LANGAN

Technical Excellence
Practical Experience
Client Responsiveness

24 October 2018

Mr. Damir Priskich
Catellus Alameda Landing Development, LLC
66 Franklin Street, Suite 200
Oakland, California 94607

Subject: Geotechnical Consultation
Light Weight Fill Material Recommendations
Alameda Landing Waterfront
Alameda, California
Project No. 731584113

Dear Mr. Priskich:

This letter presents our recommendations for the light weight fill (LWF) material for the Alameda Landing Waterfront project. This letter is prepared in response to the following request for additional information from the Engineering Criteria Review Board (ECRB) of the Bay Conservation and Development Commission (BCDC) during the meeting on 26 September 2018 and forwarded to us via e-mail on 3 October 2018:

"Provide criteria for characteristics of fill to be added landward of the wharf, including that of cellular concrete and its buoyancy potential if inundated by water."

We recommend LWF to consist of pervious and free draining material using Elastizell EF Class II (From Elastizell Corporation of America) or equivalent. The unit weight of this material should not exceed 30 pounds per cubic foot (pcf) and have an unconfined compressive strength of at least 40 pounds per square inch (psi). The use of a pervious and free draining LWF described above should be able to prevent uplift pressures against the bottom of the LWF.

We trust that this letter provides the information you require. If you have any questions, please do not hesitate to call.

Sincerely,
Langan Engineering and Environmental Services, Inc.

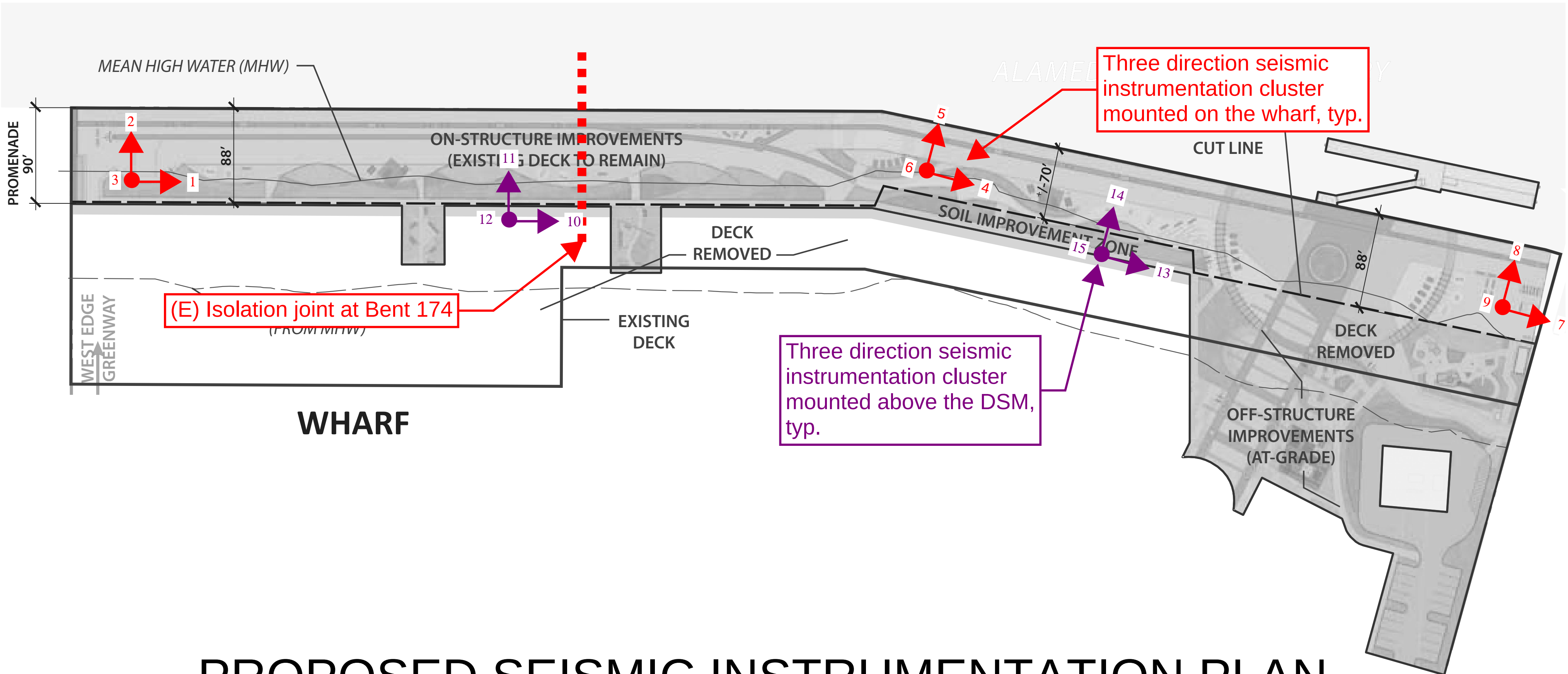
Haze M. Rodgers, PE, GE
Associate

REGISTERED PROFESSIONAL ENGINEER
HAZE M. RODGERS
NO. 2818
Exp. 06/30/20
GEOTECHNICAL
STATE OF CALIFORNIA

Ramin Golesorkhi, PhD, PE, GE
Principal/Vice President

REGISTERED PROFESSIONAL ENGINEER
RAMIN GOLESORKHI
GE 2351
Exp. 06/30/20
GEOTECHNICAL
STATE OF CALIFORNIA

731584113.12_HMR_LWF criteria



PROPOSED SEISMIC INSTRUMENTATION PLAN

ALAMEDA LANDING
WATERFRONT DEVELOPMENT PHASE

CATELLUS DEVELOPMENT CORPORATION
19 OCTOBER 2019
PREPARED BY SIMPSON GUMPERTZ & HEGER INC.

CATELLUS

ALAMEDALANDING

1 November 2018

Mr. Damir Priskich
Catellus Development Corporation
66 Franklin St., Suite 200
Oakland, CA 94607

Re: Project 177517: BCDC ECRB Comment 6, Alameda Landing Waterfront Project, Alameda, CA

Dear Mr. Priskich:

Simpson Gumpertz & Heger Inc. (SGH) is pleased to provide you with this letter in response to comments provided by BDCD's ECRB. Comment No. 6 reads as follows:

> 6. Identify sea level inundation zone and associated criteria for Wharf. Determine if Coastal Zone A is appropriate.

This letter addresses the criteria for the wharf associated with sea level inundation.

SGH has performed structural analysis of the wharf structure for a number of load conditions, including seismic loads and vertical loads uplift load on the wharf generated by waves impacting the underside of the deck.

Our wave uplift load calculations considered water levels starting with the current BFE elevation of +9.75 ft (NAVD88) and conservatively considered sea level rise up to 2.25 ft, providing a final BFE elevation of 12 ft (NAVD88). The top of the existing wharf is at an elevation of 13 ft and the deck is 2 ft thick along the waterside edge, so this 12 ft elevation corresponds roughly to the mid depth of the deck.

Using the available wind data from the nearby Alameda Naval Air Station, we determined that the maximum wave height for wind-generated waves during the 100-year storm is 1.48 ft. Through observation of the project site we determined that WETA Ferries and Coast Guard vessel passing through the estuary create the most significant waves on the existing wharf. Our calculations determined that between these two vessel classes, the maximum wave height for waves generated by passing vessels is slightly less than 1 ft. Our structural analysis of the wharf only considered the wind-generated waves as these waves are significantly larger than the waves generated by passing vessels. Note that both wave heights generally conform to the FEMA FIRM Zone AE BFE designation assigned to the project site.

Using varying levels of sea level rise and our design wave, we determined that the largest vertical uplift loads from waves impacting the underside of the deck would not overcome the weight of the deck and that seismic loads will continue to govern for the wharf structure.

We would be glad to discuss this topic further with Catellus and BDCD.

Please do not hesitate to contact me at (510) 457-4448 or by e-mail at GSJohnson@SGH.com with any further questions on this matter.

Sincerely yours,
Simpson Gumpertz & Heger Inc.

Gayle S. Johnson, P.E.
Senior Principal
CA License No. C36658

REGISTERED PROFESSIONAL ENGINEER
GAYLE S. JOHNSON
No. C36658
CIVIL
STATE OF CALIFORNIA
11/1/2018

ENGINEERS / SURVEYORS / PLANNERS

WALNUT CREEK OFFICE | 925.940.2200

MEMORANDUM

Date: 31 October 2018 **BKF Job Number:** 20165092-35

Deliver To: Bill Kennedy

Company: Catellus Development Corporation

From: Christopher C. Mills, PE

Subject: Evaluation of Alameda Landing Waterfront -- Base Flood Elevation and Area of Inundation

REMARKS:

The following summarizes our investigation of the potential depth of flooding within the Alameda Landing Waterfront property located in Alameda resulting from extreme tides and sea level rise. The project team will use this information to establish the elevations of buildings and roadways as well as to complete hydraulic analysis of the project's storm water conveyance system.

In general, the Federal Emergency Management Administration (FEMA) defines locations that are subject to inundation resulting from a storm that has a 1-percent chance of occurring in any year (sometimes known as the 100-year storm). While the 100-year storm has a low probability of occurring, the resulting rainfall can create severe inundation. FEMA defines the areas of inundation by the 1-percent-annual-chance flood event as Zone A in the federal agency's Flood Insurance Rate Maps (FIRMs). When known, the FIRM defines the Base Flood Elevation (BFE), which FEMA typically references from the North American Vertical Datum of 1988 (NAVD 88). The FIRM's relate to occupied buildings as a way of determining their insurability. Most municipalities do not allow construction within Zone A unless the applicant raises the development above the BFE.

FEMA's recent San Francisco Bay Area Coastal Study has resulted in updated FIRM's that will go into effect on 21 December 2018 and indicate that portions of the project site would be below the Zone AE BFE at an elevation of 10 (NAVD 88). While the map indicates Elevation 10, the Preliminary 2015 Flood Insurance Study on which the map is based indicates on a calculated Stillwater Elevation of 9.75 at the project site. (See pertinent pages of the study in Attachment B).

Areas that are designated AE Zones are subject to inundation by the 1-percent-annual-chance flood event. Additional hazards that could result from storm-induced velocity wave action by a higher wave at this shoreline, which fronts the Oakland Estuary, are considered insignificant.

The Authority Having Jurisdiction for the project (City of Alameda) references elevations to the City of Alameda Datum. Table 1 relates the various common datum planes in the area, including the City of Alameda's, to each other, as well as to water levels as measured at the National Oceanic and Atmospheric Administration's (NOAA) Oakland Inner Harbor Gauge (9414764).

Based upon our topographic survey of the site, existing grades are as low as about 8 feet between the existing warehouses. The wharf itself is at approximately elevation 13. The existing wharf is not in the flood zone.

Table 1.1

		Datum			
Description	**2014 MLLW[1]**	**NGVD29**	**NAVD88**	**NAS**	**Alameda**
City of Alameda Datum		3.41	6.11	*107.64*	0.00
MHHW	6.37	3.55	6.25	*107.78*	0.14
MHW	5.75	2.93	5.63	*107.16*	-0.48
MTL	3.43	0.60	3.30	*104.83*	-2.81
MSL	3.33	0.50	3.20	*104.73*	-2.91
MLW	1.11	-1.72	0.98	*102.51*	-5.13
NAVD88	0.12	-2.70	0.00	*101.53*	-6.11
MLLW	0.00	-2.82	-0.12	*101.41*	-6.23
100-Year Tide Still Water Level[2]	**9.4**	6.58	9.28		3.17
100-Year Still Water Level + 36" SLR	12.4	9.58	12.28		6.17
100-Year Still Water Level + 66" SLR	14.9	12.08	14.78		8.67

Datum Conversions

NGVD29 -- 3.41 feet = City of Alameda Datum (City of Oakland records, per Alameda city engineer, 1/7/65)
NAVD88 -- 6.11 feet = City of Alameda Datum
NAS -- 107.64 feet = City of Alameda Datum
NAVD88 -- 2.70 feet = NGVD29 (per NOAA Vertcon conversion at 37 47'30.59"N; 122 16'53.10"W)

Notes

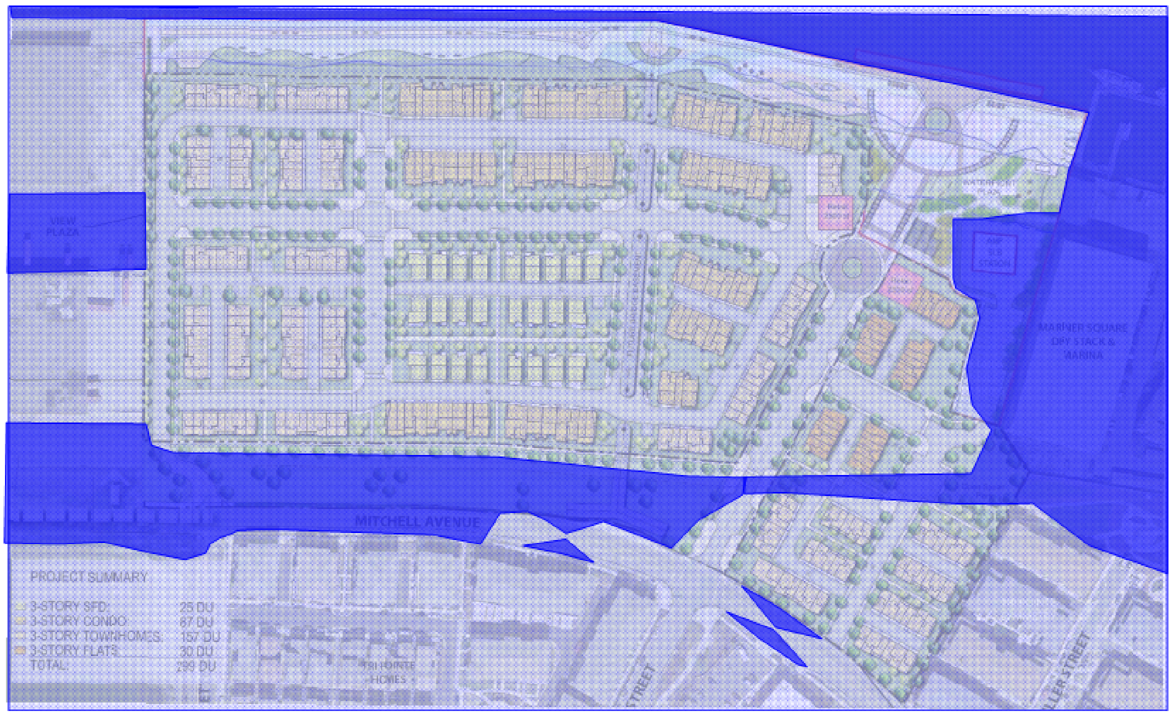
1. Per NOAA Oakland Inner Harbor gage (9414764), published 6/30/2014
2. Assumes consistent relationship of 9.4-ft between MLLW and 100 Year Tide, as reported in US Army Corps of Engineers "San Francisco Bay Tidal Stage vs. Frequency Study"; October 1984

The development of the Alameda Landing Waterfront site should also accommodate the potential increases in tide levels resulting from climate change. In a report entitled "State of California Sea-Level Rise Guidance, 2018 Update" prepared by the California Natural Resource Agency and the California Ocean Protection Council (See Attachment C), sea levels are predicted to rise to between 1.0-ft and 1.9-ft by 2070 with respect to a baseline of the average relative sea level over the period between 1991 and 2009. Predictions of sea level rise by the end of century are far less accurate. Assuming a reduction in emissions, the report indicates a 0.5% probability that sea level rise will exceed 5.7-ft by the end of the century. Since the mean higher high water tidal datum in this location is at elevation ~6 feet, regular high tides are likely to rise to an elevation of between ~7-8 feet by 2070, and there is a limited possibility that they could reach ~12 feet by the end of century.

Figure 1 (below) indicates the inundation zones that would result from sea levels rising by 1.9-ft and 5.7-ft from current BFE, respectively.

ENGINEERS / SURVEYORS / PLANNERS

FIGURE 1
INUNDATION ZONES



2050 INUNDATION (ASSUMING 1.9-ft SEA LEVEL RISE ABOVE CURRENT BFE OF 9.75 NAVD 88)

2100 INUNDATION (ASSUMING 5.7-ft SEA LEVEL RISE ABOVE CURRENT BFE OF 9.75 NAVD 88)

Based upon our discussion with staff from the City's planning and engineering departments, Alameda does not have a policy or guidelines for designing to accommodate climate change, though in March 2014 (prior to the 2018 Report referenced above) they adopted a Master Infrastructure Plan for the neighboring Alameda Point (formerly the Naval Air Station Alameda), which defined infrastructure standards for the redevelopment of the former Military Installation. The policy adopted for Alameda Point requires that new structures be constructed to maintain 1-ft of freeboard to an assumed new Base Flood Elevation assuming 24-inches of Sea Level Rise. Since FEMA does not account for rising tides due to climate change in their FIRM maps, we recommend that the project incorporate the following two-part strategy to mitigate the potential impacts of sea level rise:

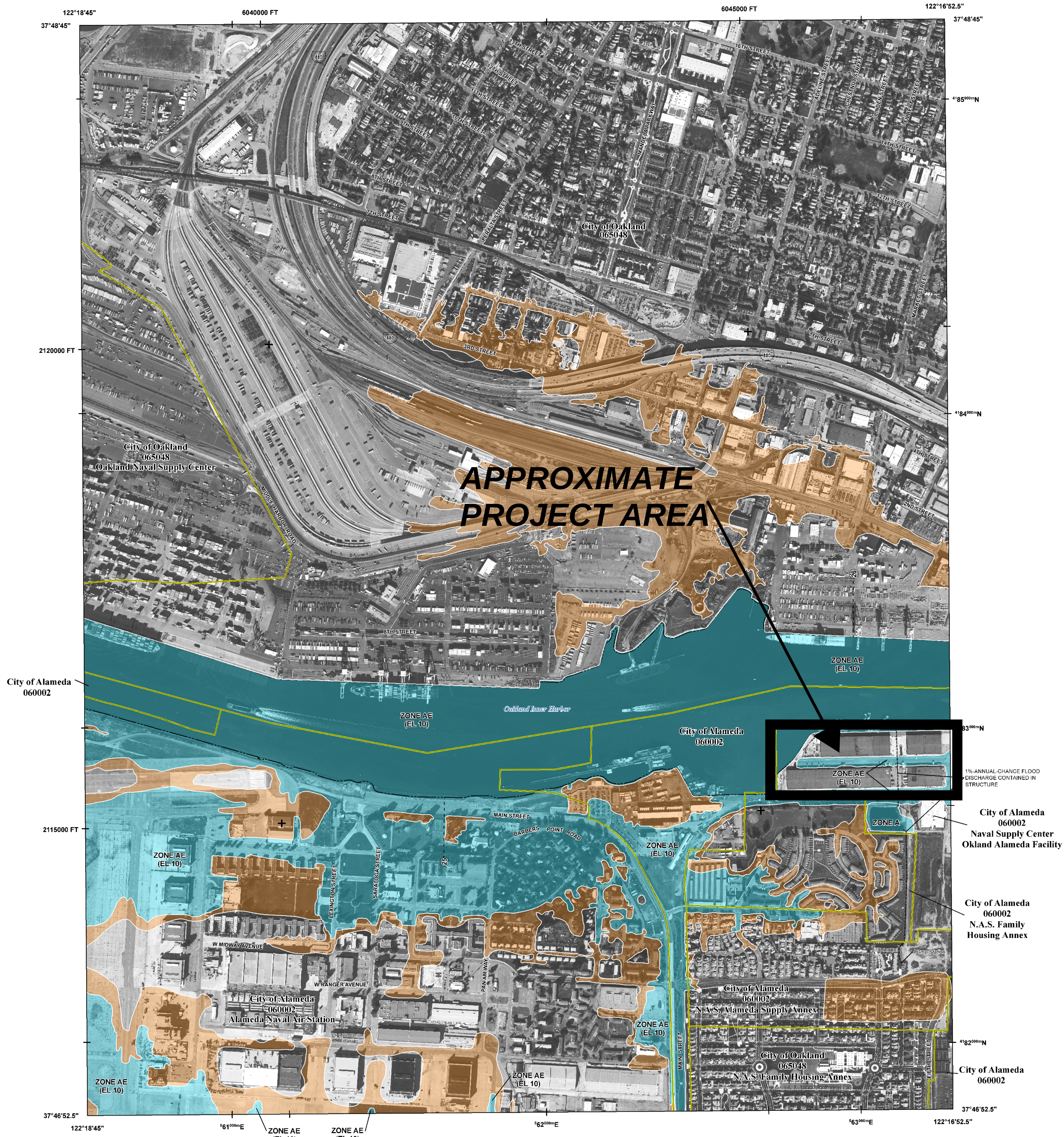
1. To address the potential for a 50-year rise in sea level of 1.9-feet inches, all buildings should be set above an adjusted BFE of 13-feet (10 feet plus 2-ft + 1-ft freeboard).
2. Since the end-of-century sea level rise is more difficult to predict and mitigation strategies are expected to evolve in the interim period, instead of raising the site to accommodate an increase in tidal elevation of more than 1.9-feet, the Alameda Landing Waterfront project should provide an area along the shoreline for an adaptive response such as a floodwall at such time as it is needed.

In summary, we note and recommend the following:

1. New structures shall accommodate 12-inches of freeboard above the 1.9-ft of sea level rise estimated to occur between 2000 and 2070 by the California National Resources Agency and California Ocean Protection Council in their 2018 Update of the State of California Sea-Level Rise Guidance Document, assuming high emissions, and the low risk aversion / likely range scenario. Finished floors of new structures shall be a minimum of 36-inches above current Base Flood Elevation, as defined by FEMA's latest Flood Insurance Study.
2. Set all residential buildings to a finished floor elevation of 36-inches above the current BFE at 9.8 feet. Thus, the finished floor elevation of all buildings should be set at or above 12.8 feet.
3. For hydraulic calculations, set the tail water elevation to mean higher high water or 6.3 feet.

If there are any questions or comments, please contact Christopher Mills at 925.940.2207 or cmills@bkf.com.

ATTACHMENT #1 (PAGE 1 OF 2)



FLOOD HAZARD INFORMATION

SEE FIS REPORT FOR DETAILED LEGEND AND INDEX MAP FOR FIRM PANEL LAYOUT

THE INFORMATION DEPICTED ON THIS MAP AND SUPPORTING DOCUMENTATION ARE ALSO AVAILABLE IN DIGITAL FORMAT AT

HTTP://MSC.FEMA.GOV

SPECIAL FLOOD HAZARD AREAS
- Without Base Flood Elevation (BFE) *Zone A, V, A99*
- With BFE or Depth *Zone AE, AO, AH, VE, AR*
- Regulatory Floodway

OTHER AREAS OF FLOOD HAZARD
- 0.2% Annual Chance Flood Hazard, Areas of 1% annual chance flood with average depth less than one foot or with drainage areas of less than one square mile *Zone X*
- Future Conditions 1% Annual Chance Flood Hazard *Zone X*
- Area with Reduced Flood Risk due to Levee See Notes. *Zone X*
- Area with Flood Risk due to Levee *Zone D*

OTHER AREAS
- NO SCREEN Area of Minimal Flood Hazard *Zone X*
- Area of Undetermined Flood Hazard *Zone D*

GENERAL STRUCTURES
- Channel, Culvert, or Storm Sewer
- Levee, Dike, or Floodwall

OTHER FEATURES
- E 18.2 17.5 Cross Sections with 1% Annual Chance Water Surface Elevation
- 8 Coastal Transect
- Coastal Transect Baseline
- Profile Baseline
- Hydrographic Feature
- 513 Base Flood Elevation Line (BFE)
- Limit of Study
- Jurisdiction Boundary

NOTES TO USERS

For information and questions about this map, available products associated with this FIRM including historic versions of this FIRM, how to order products or the National Flood Insurance Program in general, please call the FEMA Map Information eXchange at 1-877-FEMA-MAP (1-877-336-2627) or visit the FEMA Map Service Center website at http://msc.fema.gov. Available products may include previously issued Letters of Map Change, a Flood Insurance Study Report, and/or digital versions of this map. Many of these products can be ordered or obtained directly from the website. Users may determine the current map date for each FIRM panel by visiting the FEMA Map Service Center website or by calling the FEMA Map Information eXchange.

Communities annexing land on adjacent FIRM panels must obtain a current copy of the adjacent panel as well as the current FIRM Index. These may be ordered directly from the Map Service Center at the number listed above.

For community and countywide map dates refer to the Flood Insurance Study report for this jurisdiction.

To determine if flood insurance is available in this community, contact your Insurance agent or call the National Flood Insurance Program at 1-800-638-6620.

Base map information shown on this FIRM was derived from Coastal California LiDAR and Digital Imagery dated 2011. USDA NAIP 2012 imagery is used in areas not covered by the Coastal California imagery.

SCALE

N

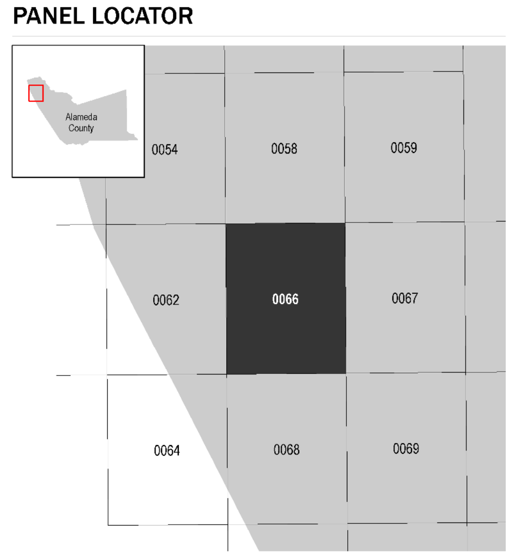
Map Projection:
Universal Transverse Mercator Zone 10N; North American Datum 1983; Western Hemisphere; Vertical Datum: NAVD 88

1 inch = 500 feet 1:6,000

0 250 500 750 1,000 2,000 Feet

0 125 250 500 Meters

PANEL LOCATOR



FEMA

National Flood Insurance Program

NATIONAL FLOOD INSURANCE PROGRAM

FLOOD INSURANCE RATE MAP

ALAMEDA COUNTY, CALIFORNIA

and Incorporated Areas

PANEL 66 OF 725

FEMA

Panel Contains:

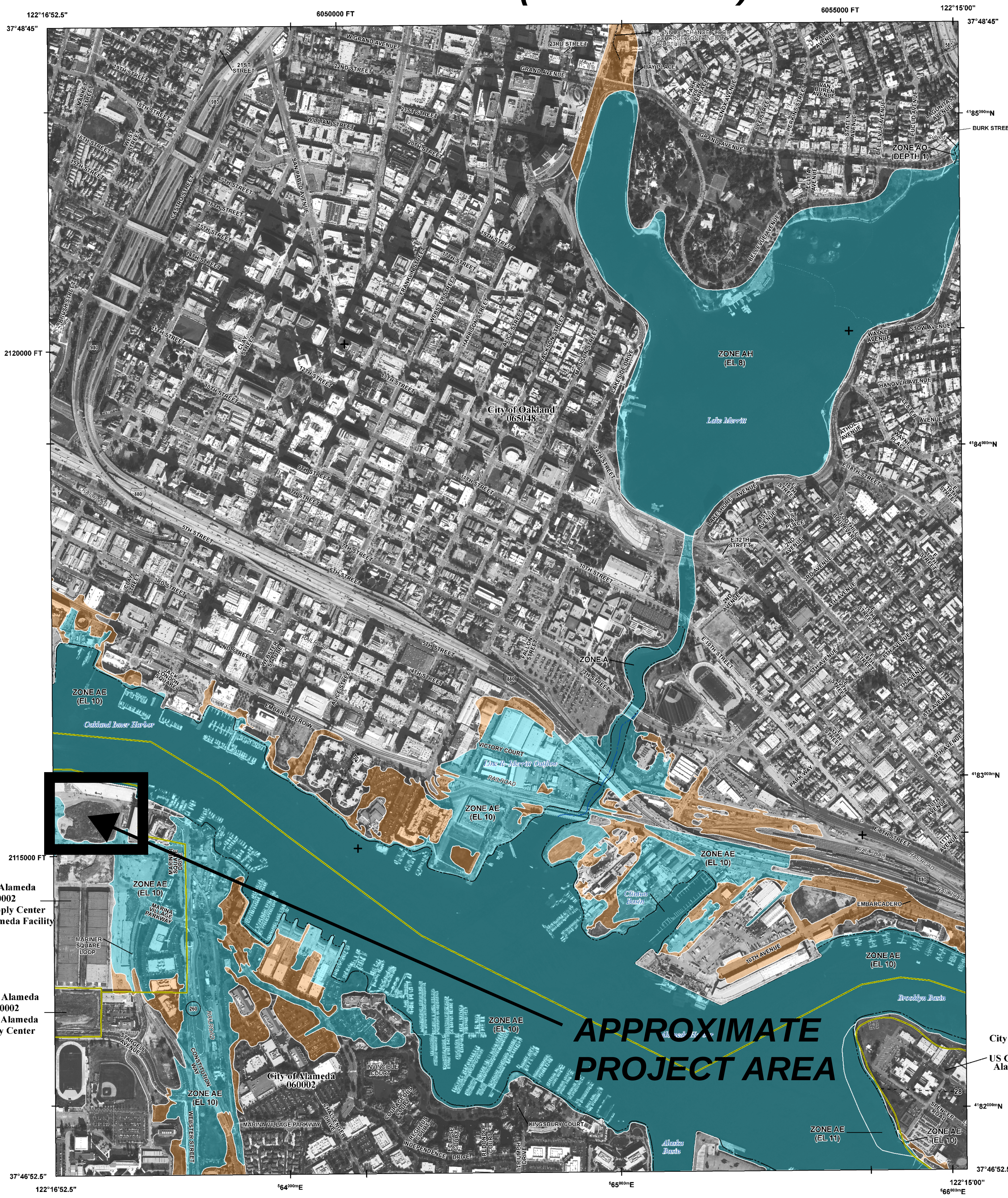
COMMUNITY	NUMBER	PANEL	SUFFIX
ALAMEDA, CITY OF	060002	0066	H
OAKLAND, CITY OF	065048	0066	H

VERSION NUMBER
2.3.2.0

MAP NUMBER
06001C0066H

MAP REVISED
DECEMBER 21, 2018

ATTACHMENT #1 (PAGE 2 OF 2)



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	Regulatory Floodway
OTHER AREAS OF FLOOD HAZARD	0.2% Annual Chance Flood Hazard, Areas of 1% annual chance flood with average depth less than one foot or with drainage areas of less than one square mile *Zone X*
	Future Conditions 1% Annual Chance Flood Hazard *Zone X*
	Area with Reduced Flood Risk due to Levee See Notes. *Zone X*
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OTHER AREAS	NO SCREEN Area of Minimal Flood Hazard *Zone X*
	Area of Undetermined Flood Hazard *Zone D*
GENERAL STRUCTURES	Channel, Culvert, or Storm Sewer
	Levee, Dike, or Floodwall
	E 18.2 17.5 Cross Sections with 1% Annual Chance Water Surface Elevation
	8 Coastal Transect
	Coastal Transect Baseline
	Profile Baseline
	Hydrographic Feature
	513 Base Flood Elevation Line (BFE)
OTHER FEATURES	Limit of Study
	Jurisdiction Boundary

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SCALE

N

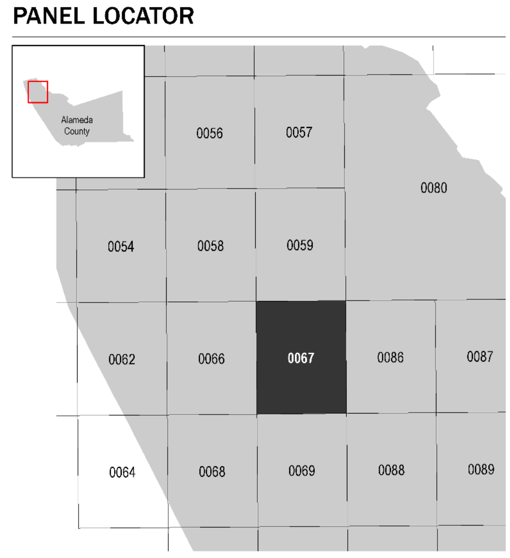
Map Projection:
Universal Transverse Mercator Zone 10N; North American Datum 1983; Western Hemisphere; Vertical Datum: NAVD 88

1 inch = 500 feet 1:6,000

0 250 500 750 1,000 2,000 Feet

0 125 250 500 Meters

PANEL LOCATOR



FEMA

National Flood Insurance Program

NATIONAL FLOOD INSURANCE PROGRAM
FLOOD INSURANCE RATE MAP

ALAMEDA COUNTY, CALIFORNIA
and Incorporated Areas

PANEL 67 OF 725

FEMA

Panel Contains:

COMMUNITY	NUMBER	PANEL	SUFFIX
ALAMEDA, CITY OF	060002	0067	H
OAKLAND, CITY OF	065048	0067	H

VERSION NUMBER
2.3.2.0

MAP NUMBER
06001C0067H

MAP REVISED
DECEMBER 21, 2018

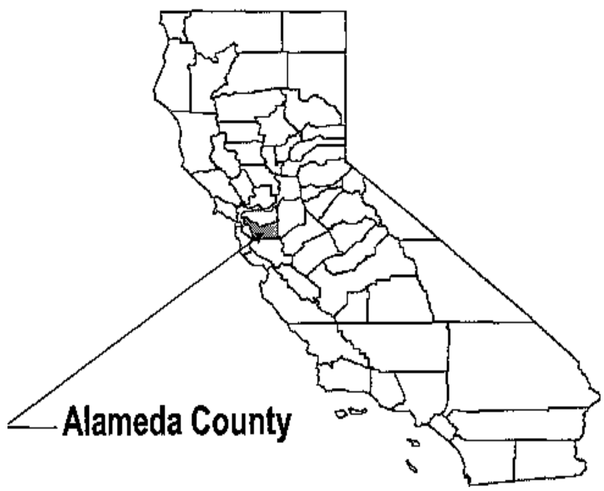
FLOOD INSURANCE STUDY

VOLUME 1 OF 3

ALAMEDA COUNTY, CALIFORNIA
AND INCORPORATED AREAS

Community Name	Community Number
ALAMEDA COUNTY (UNINCORPORATED AREAS)	060001
ALAMEDA, CITY OF	060002
ALBANY, CITY OF	060003
BERKELEY, CITY OF	060004
DUBLIN, CITY OF	060705
EMERYVILLE, CITY OF	060005
FREMONT, CITY OF	065028
HAYWARD, CITY OF	065033
LIVERMORE, CITY OF	060008
NEWARK, CITY OF	060009
OAKLAND, CITY OF	065048
* PIEDMONT, CITY OF	060011
PLEASANTON, CITY OF	060012
SAN LEANDRO, CITY OF	060013
UNION CITY, CITY OF	060014

* Non Flood – Prone Community



PRELIMINARY
4/16/2015

REVISED
Month Day, Year

Federal Emergency Management Agency

FLOOD INSURANCE STUDY NUMBER
06001CV001B

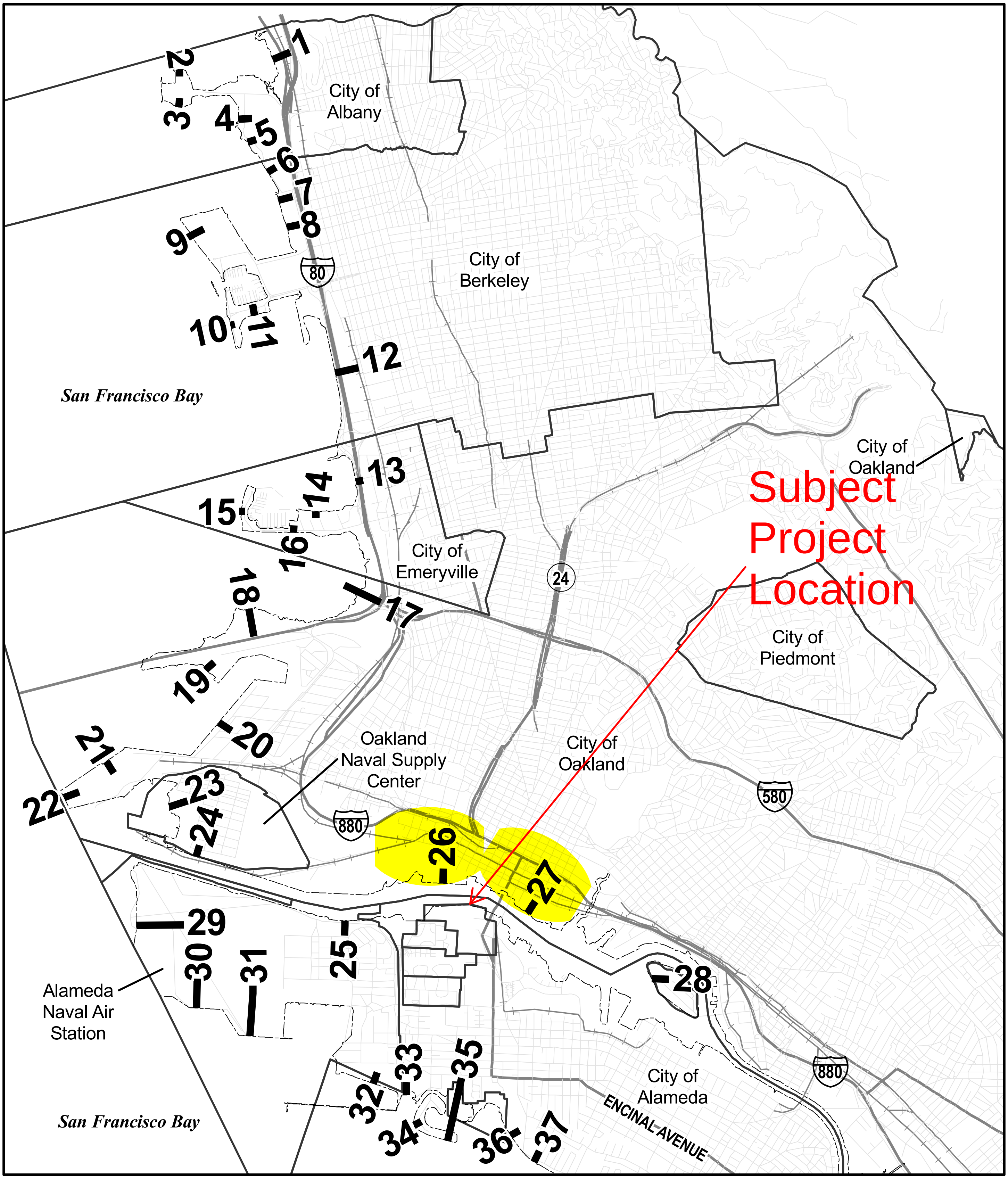


FIGURE 2

FEDERAL EMERGENCY MANAGEMENT AGENCY

ALAMEDA COUNTY, CALIFORNIA AND INCORPORATED AREAS

0 0.4 0.8 1.6 2.4 Miles

N

TRANSECT LOCATION MAP #1

Attachment B (Page 3 of 3)

Transect Data

Transect	XY Coordinates (Latitude/Longitude)		Stillwater Elevation (feet NAVD 88)[1]				Zone	BFE
			10% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance		
1	37.893671	-122.310885	8.5	9.41	9.96	11.46	VE	13[2]
2	37.892438	-122.325067	8.49	9.4	9.95	11.43	AE	10
3	37.887911	-122.325163	8.49	9.39	9.94	11.4	VE	14[2]
4	37.886473	-122.316068	8.49	9.39	9.94	11.41	VE	15[2]
5	37.883732	-122.314709	8.49	9.38	9.93	11.39	VE	16[2]
6	37.880127	-122.311621	8.48	9.39	9.92	11.37	VE	13[2]
7	37.876770	-122.309976	8.48	9.39	9.91	11.37	VE AE	13[2] 10
8	37.873520	-122.308704	8.48	9.37	9.92	11.38	VE	14[2]
9	37.872532	-122.323127	8.48	9.34	9.9	11.35	VE	14[2]
10	37.861738	-122.317158	8.48	9.33	9.89	11.31	VE	13[2]
11	37.862927	-122.313532	8.49	9.33	9.9	11.32	VE	12[2]
12	37.856066	-122.301282	8.48	9.32	9.89	11.3	VE AE	13[2] 10
13	37.843024	-122.298111	8.47	9.35	9.88	11.3	AE	12[2]
14	37.839449	-122.304161	8.47	9.37	9.88	11.3	AE	11[2]
15	37.839403	-122.315446	8.47	9.36	9.86	11.24	VE	13[2]
16	37.836885	-122.307495	8.48	9.34	9.86	11.24	AE	11[2]
17	37.829932	-122.298067	8.48	9.33	9.87	11.25	AE	10-11
18	37.827714	-122.314246	8.47	9.32	9.84	11.21	AE	11
19	37.820368	-122.320764	8.5	9.38	9.76	10.94	VE AE	11 10
20	37.814058	-122.318750	8.49	9.49	9.76	10.93	VE	12[2]
21	37.809322	-122.335738	8.5	9.48	9.76	10.93	VE	12[2]
22	37.804945	-122.342126	8.52	9.48	9.71	10.77	VE	16[2]
23	37.803909	-122.326182	8.55	9.7	9.75	10.83	AE	12[2]
24	37.797971	-122.322093	8.54	9.69	9.79	10.96	AE	10
25	37.790292	-122.299529	8.52	9.7	9.78	10.97	AE	10[2]
26	37.794817	-122.284720	8.52	9.74	9.76	10.91	AE	10
27	37.791244	-122.271908	8.52	9.74	9.73	10.84	AE	10
28	37.783461	-122.253095	8.52	9.75	9.73	10.83	AE	11[2]
29	37.789674	-122.331089	8.58	9.75	9.78	10.86	VE AE	12[2] 10
30	37.779764	-122.321968	8.67	9.74	9.91	11.07	VE	12[2]
31	37.776414	-122.313868	8.67	9.57	9.9	11.05	VE AE	12[2] 10
32	37.770764	-122.295181	8.67	9.43	9.9	11.05	VE	11[2]
33	37.769404	-122.290194	8.83	9.41	10.15	11.49	VE	11[2]

Project Area is between Transects 26 and 27. Use 9.75-ft as conservative interpolation

ATTACHMENT C

TABLE 1: Projected Sea-Level Rise (in feet) for San Francisco

Probabilistic projections for the height of sea-level rise shown below, along with the H++ scenario (depicted in blue in the far right column), as seen in the Rising Seas Report. The H++ projection is a single scenario and does not have an associated likelihood of occurrence as do the probabilistic projections. Probabilistic projections are with respect to a baseline of the year 2000, or more specifically the average relative sea level over 1991 - 2009. High emissions represents RCP 8.5; low emissions represents RCP 2.6. ***Recommended projections for use in low, medium-high and extreme risk aversion decisions are outlined in blue boxes below.***

		Probabilistic Projections (in feet) (based on Kopp et al. 2014)					H++ scenario (Sweet et al. 2017) *Single scenario
		MEDIAN	LIKELY RANGE		1-IN-20 CHANCE	1-IN-200 CHANCE	
		50% probability sea-level rise meets or exceeds…	66% probability sea-level rise is between…		5% probability sea-level rise meets or exceeds…	0.5% probability sea-level rise meets or exceeds…	
				Low Risk Aversion		Medium - High Risk Aversion	Extreme Risk Aversion
High emissions	2030	0.4	0.3 -	0.5	0.6	0.8	1.0
	2040	0.6	0.5 -	0.8	1.0	1.3	1.8
	2050	0.9	0.6 -	1.1	1.4	1.9	2.7
Low emissions	2060	1.0	0.6 -	1.3	1.6	2.4	
High emissions	2060	1.1	0.8 -	1.5	1.8	2.6	3.9
Low emissions	2070	1.1	0.8 -	1.5	1.9	3.1	
High emissions	2070	1.4	1.0 -	1.9	2.4	3.5	5.2
Low emissions	2080	1.3	0.9 -	1.8	2.3	3.9	
High emissions	2080	1.7	1.2 -	2.4	3.0	4.5	6.6
Low emissions	2090	1.4	1.0 -	2.1	2.8	4.7	
High emissions	2090	2.1	1.4 -	2.9	3.6	5.6	8.3
Low emissions	2100	1.6	1.0 -	2.4	3.2	5.7	
High emissions	2100	2.5	1.6 -	3.4	4.4	6.9	10.2
Low emissions	2110*	1.7	1.2 -	2.5	3.4	6.3	
High emissions	2110*	2.6	1.9 -	3.5	4.5	7.3	11.9
Low emissions	2120	1.9	1.2 -	2.8	3.9	7.4	
High emissions	2120	3	2.2 -	4.1	5.2	8.6	14.2
Low emissions	2130	2.1	1.3 -	3.1	4.4	8.5	
High emissions	2130	3.3	2.4 -	4.6	6.0	10.0	16.6
Low emissions	2140	2.2	1.3 -	3.4	4.9	9.7	
High emissions	2140	3.7	2.6 -	5.2	6.8	11.4	19.1
Low emissions	2150	2.4	1.3 -	3.8	5.5	11.0	
High emissions	2150	4.1	2.8 -	5.8	5.7	13.0	21.9

**Most of the available climate model experiments do not extend beyond 2100. The resulting reduction in model availability causes a small dip in projections between 2100 and 2110, as well as a shift in uncertainty estimates (see Kopp et al. 2014). Use of 2110 projections should be done with caution and with acknowledgement of increased uncertainty around these projections.*

113 Cooper Street
Santa Cruz, CA 95060

31 October 2018

Dr. Juan Baez
President
Advanced GeoSolutions, Inc.
13 Orchard Road, Suite 105
Lake Forest CA 92630

via email:
jibaez@advgeosolutions.com

Re: Alameda Landing, Waterfront Development Phase
Reply to questions posed by the Engineering Criteria Review Board
Geotechnical Earthquake Engineering applications

Dear Dr. Baez,

On behalf of the Alameda Landing Waterfront Development project team, we appreciate the opportunity to further explain, and hopefully clarify, for the BCDC and Engineering Criteria Review Board (ECRB) the basis for geotechnical earthquake engineering analyses performed in support of the project.

BCDC Questions

It is our understanding that a series of questions was initially prepared by the ECRB following the project hearing held at BCDC on Wednesday September 26, 2018, and that these questions were forwarded to Catellus on October 3, 2018. We received the questions via email on October 18, 2018 and we are pleased to submit our replies to the following questions;

1. *Develop estimates of relative displacements induced by wave passage effect using appropriate MCE time histories for Hayward and San Andreas faults. Determine if seismic joint criteria are consistent with anticipated wave-passage displacements.*
2. *Introduce new notation to refer to average interval shear velocities in bedrock, by designating the depth interval as indicated for an interval of 45 m to 60 m by "Vs45-60". This change in notation is needed to eliminate confusion introduced*

by incorrectly referring to the bedrock interval velocities using the notation Vs30.

The following additional question was posed by the BCDC via email on October 24;

3. *MCE time histories appropriate for the Hayward Fault and the San Andreas Fault.*

Response to Questions

Wave Passage Effects on Wharf Performance

The ECRB has requested that the project team address the potential impact of near-surface ground motion variation due to Wave Passage Effects on the seismic performance of the wharf. We have interpreted Wave Passage Effects to represent a form of spatial incoherence in the seismic motions that is due to nonvertical waves reaching different points of the wharf at different times, producing a time shift between the motions at these points. This effect is considered separate from incoherence due to factors such as extended source effects, ray-path effects, or wave scattering (e.g., Kramer 1996).

The potential influence of wave passage on the seismic performance of the wharf is related to the longest continuous dimension of the wharf (i.e., between construction joints, isolation joints, or shear keys) relative to the ground motion wave length of interest. The length of interest will vary in the transverse and longitudinal direction of the wharf. For example, it is expected that the transverse dimension of the modified wharf (68 ft to 88 ft) is short enough that in this direction the ground motions are virtually the same at the waterfront and landward ends of the structure, thus wave passage effects are insignificant. This can be simply demonstrated by applying a time shift in the arrival of the seismic waves at each support (pile), or at each end of the wharf. An apparent wave speed of 8,250 ft/sec is commonly recommended in U.S. highway practice for long-span bridges (Kavazanjian et. al. 2011). This practice-oriented approximation suggests that the time shift in the displacement time history should be roughly 0.011 seconds (88 ft / 8,250 fps). The change in transient displacement would be quite small during the 0.011 sec time-step. The time shift in the transverse direction will be greater, which is described as follows.

The distance between isolation joints in the wharf is approximately 450 ft to the west of the isolation joint, which is at Bent 174 (Property Line with Bay Ship and Yacht is at Bent 129), and approximately 875 ft to the east of the expansion joint. The time shift using the greater length is therefore 0.11 sec.

The relative displacement between two points separated by 875 ft can be approximated by inspection of the displacement time histories computed at the top of the Old Bay Clay (OBC) using MCE level input motions in the 1-D site response analysis for free-field conditions. The average maximum displacement computed at the top of the OBC was 1.71 ft. The resulting average "relative displacement" estimated by applying the 0.11 sec time shift is 0.31 ft (3.7 in). In addition, the computed relative displacement using the acceleration time history representative of the M 7.3 Hayward Fault scenario was only slightly greater (3.8 in).

These relative displacements approximating the wave passage effects are considered insignificant when compared to the transient deck displacement associated with inertial loading of the wharf and permanent displacement of the foundation soils that is accumulated during strong shaking (i.e. displacement demand). This observation is consistent with U.S. bridge design guidelines that indicate that wave passage effects are generally not believed to be significant in bridges less than 1,500 ft long (Marsh et. al. 2011)

The project team also shares, for consideration by the ECRB, a pertinent, local case study involving the seismic performance of a pile-supported wharf subjected to moderate ground motions (PGA ≈ 0.30 g) during the 1989 M 6.9 Loma Prieta Earthquake that has been evaluated by a member of the project team (Donahue et. al. 2005). The recorded free-field and structural motions at Berth 24/25 at the Port of Oakland were used to evaluate the response of the wharf to seismic loading, which inherently included wave passage effects. It is important to note that Berth 24/25 was not substantially affected during this earthquake, due in large part to the very limited extent of liquefiable soils in the foundation and small permanent deformations.

One objective of the investigation was to evaluate the influence of wave passage effects on induced torsion of the wharf. Torsion may be induced by incoherent motions between different sections of the wharf due to the "wave passage" effect of seismic energy as it moves past the long wharf structure. Strong motion data from the 12-channel array was used in empirical and 3-D numerical analyses of the wharf. The maximum transient displacement of the wharf deck was roughly 10 cm (3.93 in); however, the maximum relative transient displacement of the ends of the approximately 1,500 ft long section of the structural instrumentation array was 4.2 cm (1.65 in), demonstrating negligible torsion and a low possibility of resulting damage of the wharf or piles solely due to this response. We feel that this local case study provides useful guidance for bracketing the likely range of relative displacement of a pile supported wharf due to wave passage effects for the ground motions experienced during the 1989 Loma Prieta Earthquake. We acknowledge that these ground motions are less than the currently defined MCE-level motions; however, it is our opinion that the approximation outlined herein demonstrates that wave passage effects are adequately covered in the conservative analyses that have been applied for the wharf.

Wave passage effects are generally not applied in the seismic design of wharves as they can be considered short-span structures, with significant inherent redundancies that create a more uniform inertial response under seismic excitation as the energy is absorbed nearly continuously along the structure. In addition, the relative ground displacement between pile bents due only to wave passage is very small and can be accommodated by typical wharf "strong beam – weak column" design. It may be helpful to note that the ASCE COPRI 61-14 Standard "Seismic Design of Piers and Wharves" does not address wave passage as a consideration for commercial port structures.

Shear Wave Velocity in Bedrock

Bedrock shear wave velocity (V_s) is a requisite parameter for both; (i) the Ground Motion Prediction Equations (GMPE's), or Ground Motion Models (GMM's), used for ground motion characterization in PSHA and DSHA, and (ii) dynamic soil response analyses where it is used to define the low-strain shear stiffness of the base layer, or transmitting boundary, and establishes the Impedance Contrast at the base of the soil profile. The bedrock ground motions used as the base excitation in numerical soil response analyses are scaled to closely match the target acceleration response spectra (ARS) derived from the SHA for a specified time-averaged shear wave velocity over a depth interval of 30 m (100 ft), therefore it is imperative that the bedrock shear wave velocity used in the dynamic soil response analysis is consistent with the value used in the SHA. For the Alameda Landing project, we have used a time-averaged shear wave velocity of 4,000 ft/sec (1,220 m/sec) in both the SHA and site response analyses. This Vs value is considered representative of local Franciscan Formation bedrock in the depth range of interest beneath the proposed development (600 to 800 ft) based on our collection of data from numerous Vs – Vp suspension logging investigations conducted in this portion of the San Francisco Bay Area.

It was appropriately noted by Dr. Borcherdt (Chair, ECRB) that the shear wave velocity, Vs30, routinely applied in GMPE's and in seismic codes and standards (e.g., ASCE COPRI 61, ASCE 7) as the basis for defining the Site Class is defined for the time-averaged value of Vs measured from the ground surface to the depth of 30 m, therefore it represents the interval of Vs0-30, with Vs30 used as the common shorthand notation. We concur that the Vs0-30 is appropriate for use in seismic codes and GMPE's when estimating ground surface motions.

The dynamic site response analyses (both 1D and 2D); however, require input motions that are representative of the low-strain shear stiffness at the base of the model. Therefore the depth interval over which Vs is averaged varies. For the sake of our bedrock ground motion characterization and 1D site response analysis we estimated Vs from the top of Franciscan bedrock to an elevation that was 100 ft beneath the top of rock, thereby maintaining consistency with the development of the GMPE's used in the SHA. The project-specific depth interval in the Franciscan bedrock was therefore 700 ft to 800 ft (213 m to 244 m). The ECRB has recommended, for the sake of clarity, that the project team refer to this interval as Vs213-244 in order to distinguish it from Vs0-30. It has been acknowledged by the project team and ECRB that the representative time-averaged shear wave velocity in Franciscan bedrock will substantially vary between these two depth intervals. The project team will make this amendment to the Vs depth interval notation in all future work products. We wish to confirm for the ECRB and BCDC that the clarification in Vs notation does not change the input parameters or results of the dynamic soil response and deformation analyses performed to date.

MCE Time Histories

The selection and scaling of the bedrock time histories used in the 1-D dynamic soil response analysis has been outlined by Atlas Geotechnical, Inc. in Appendix J "Site Response Analysis and MCE Level Ground Motions" of the DRAFT design submittal prepared by AGI (23 August 2018). As addressed in the Atlas Geotechnical document, five of the bedrock motions were selected to be representative of the regional seismic hazard and consistent with the Uniform Hazard Spectrum for motions having a 2% probability of exceedance in 50 years (MCE) obtained from the regional PSHA. Two additional bedrock motions were selected to be representative of pertinent scenario earthquakes based on the PSHA deaggreagtion; (i) M 7.3 Hayward Fault, and (ii) M 8.0 San Andreas Fault (along the north peninsula segment of the fault).

The criteria for selecting the acceleration time histories considered the following aspects of the bedrock ground motions; spectral content and amplitude both before and after scaling (i.e., amplitude and frequency content), significant duration, energy as represented by Arias Intensity, and near-fault characteristics (e.g., pulse motions).

It is our opinion that the suite of bedrock motions representative of the MCE time histories is appropriate for the Hayward Fault and the San Andreas Fault.

References

1. Advanced Geosolutions, Inc. (2018). Soil-Foundation-Structure-Interaction Analysis for the Existing Wharf Structure and Ground Improvement by Deep Soil Mixing Buttress, Alameda Landing Waterfront Development, Alameda, California, Submittal No. 01 – Rev 02, Design Submittal, DRAFT, dated August 23, 2018.
2. Donahue, M. J., Dickenson, S. E., Miller, T. H., and Yim, S. C. (2005). Implications of the Observed Seismic Performance of a Pile-Supported Wharf for Numerical Modeling, Earthquake Spectra, 31(3), August, 617 – 634, EERI.
3. Kavazanjian, E., Wang, J.-N., Martin, G. R., Shamsabadi, A., Lam, P., Dickenson, S. E., and Hung, C. J. (2011). LRFD Seismic Analysis and Design of Transportation Geotechnical Features and Structural Foundations, Geotechnical Engineering Circular No. 3, Publication FHWA-NHI-11-032, U.S. Dept. of Transportation, FHWA, NHI.
4. Kramer, S. L. (1996). Geotechnical Earthquake Engineering, Prentice Hall, Inc., 653 pages.
5. Marsh, L., Buckle, I. G., Imbsen, R.A., and Kavazanjian, E. (2011). LRFD Seismic Analysis and Design of Bridges, Reference Manual, Publication No. FHWA-NHI-11-030, U.S. Dept. of Transportation, FHWA, NHI.

We hope that these explanations are helpful for the ECRB and BCDC staff as you continue your efforts associated with the Alameda Landing Waterfront Development project. If you have any questions pertaining to the geotechnical earthquake engineering applications on this project please do not hesitate to contact us directly.

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Memo

From:	Juan Baez	**To:**	Damir Priskich
Date:	October 29th, 2018	**CC:**	Haze Rodgers (Langan Associates)

Re: **Response to ECRB Comments re Meeting on Nov 13th, 2018**

Alameda Waterfront Project

Damir,

This Memo addresses Advanced Geosolutions Inc (AGI) response to comments/question by the ECRB as indicated in BCDC e-mail communication of October 24th, 2018

In our opinion, questions 1 and 6 (re-stated below) are related and can be addressed in a single response. The questions were:

1. The impact of lateral movement of the fill associated with the bay mud, and its impact on the wharf.
6. The ECRB had not seen a plan for the DSM. It is going to be a densified remediation zone to improve the soil response, right next to the wharf. How is it going to be affected by the lateral movement of the bay mud beneath it or close to it, and what is going to be the effect of the lateral movement on the wharf?

AGI Response:

The DSM plan and column layout has been previously included in AGI's Design Submittal of September 10th, 2018 (Appendix D –Page 171 of 319). Similarly, the worst case loading condition involving a superposition of Fill, Structural Loads from building pads, and MCE ground motions has been evaluated to assess lateral movements on Bay Mud and displacement magnitude on the wharf foundations. The output of these analyses are included in Appendixes N, O, and P of the referenced AGI design submittal.

We note that, although the referenced loading condition constitutes the anticipated highest loads imparting lateral displacement on Bay Mud and wharf supported piles, the following Table outlines the estimated lateral deformations resulting from the construction stages and application of loads at various times. The attached Appendixes A, and B, depict the Finite Element Plaxis output for the critical cross sections D-2 and F2, respectively, as referenced in the AGI design report.

Loading Case No	Description	Cross Section D-2, Displacement on Bay Mud (inches)	Cross Section D-2, Displacement on Wharf Pile Row J (inches)	Cross Section F-2, Displacement on Bay Mud (inches)	Cross Section F-2, Displacement on Wharf Pile Row J (inches)
1	During DSM Installation from EL +3ft [1]	< 1	< 1	< 1	< 1
2	After placement of Fill from EL +3ft to +8FT	< 1	< 1	< 1	< 1
3	After placement of Building Loads (450 PSF)	< 1	< 1	< 1	< 1
4	After design MCE events [2]	12	13	12	13

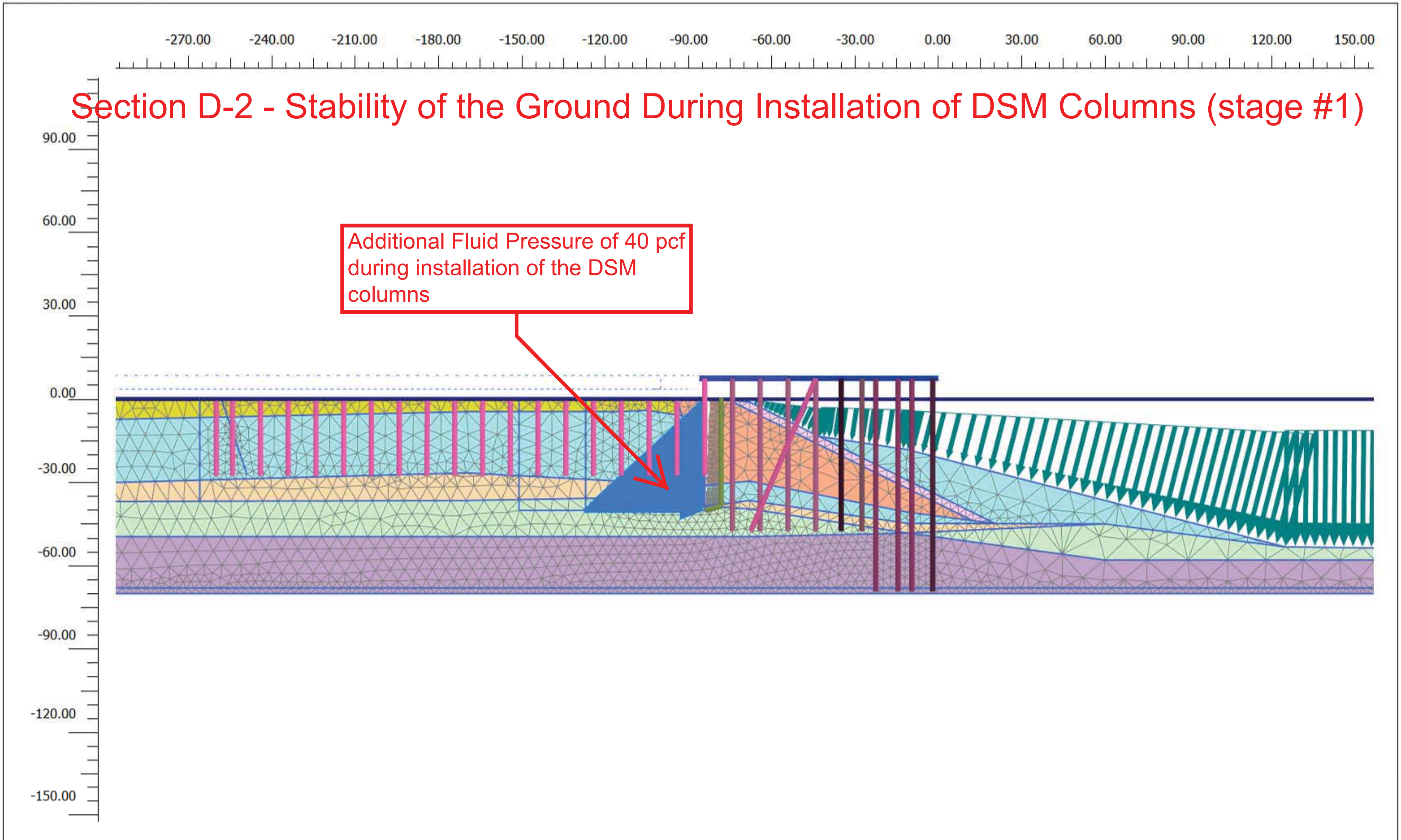
Notes:

1- During DSM installations, the blended ground experiences a temporary increased in lateral pressure resulting from the placement of fluid grout. The total fluid pressure is estimated to be up to 102 PCF (grout unit weight) and the net additional fluid pressure experienced by the surrounding soil is on the order of 40 PCF. The highest lateral fluid pressure loads are anticipated to last less than 4 hours and, as the DSM column gains strength and soil-cement reaches a plastic state, the imposed pressures are reduced over time.

2- Average displacement resulting from 7 ground motions with +x and –x directions.

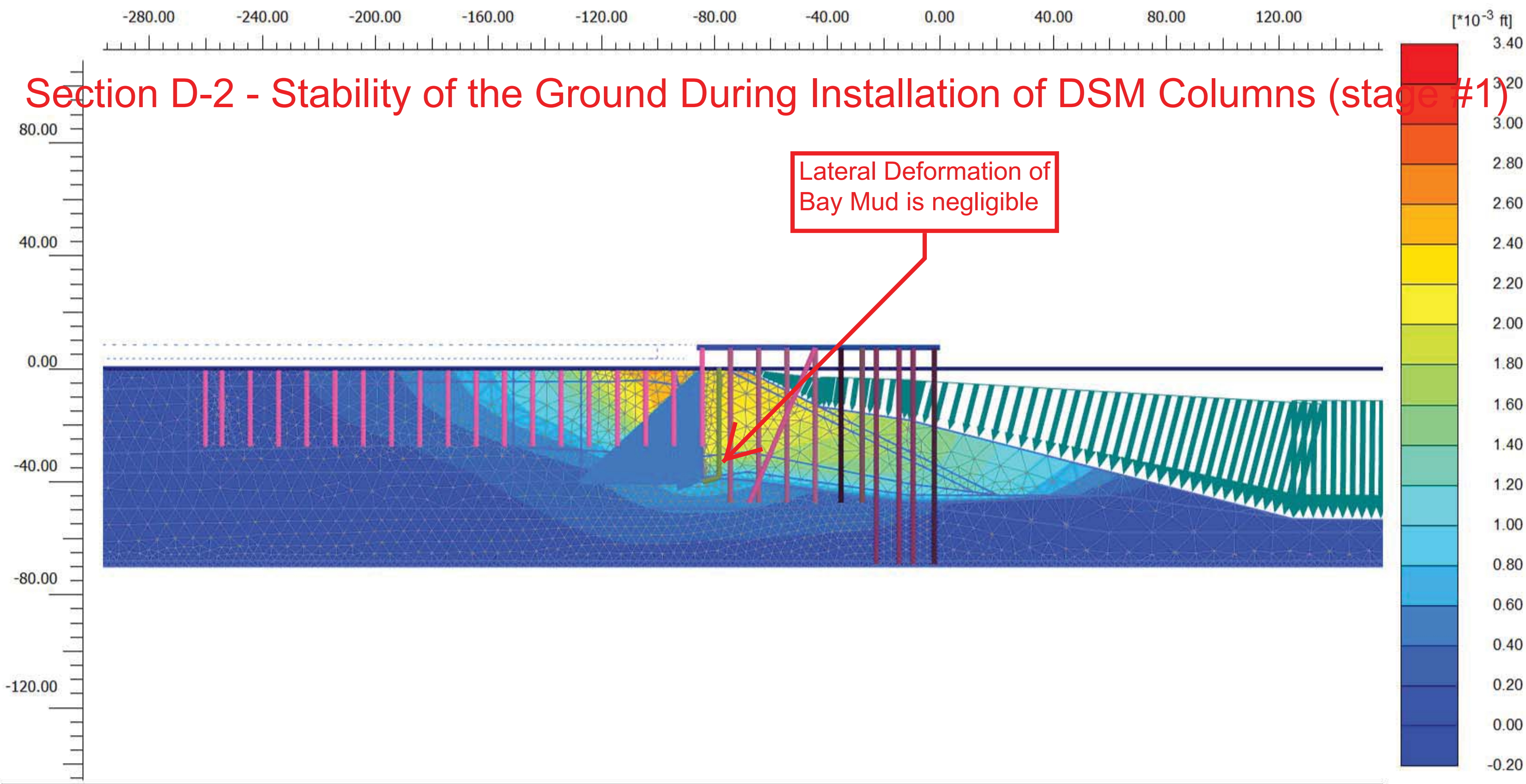
APPENDIX A

Output Version 2018.0.0.0



AGI	Project description		Date 10/21/2018
	Project filename Section D-2_Stage1	Step 8	User name Advanced Geosolutions Inc (AGI)

Output Version 2018.0.0.0



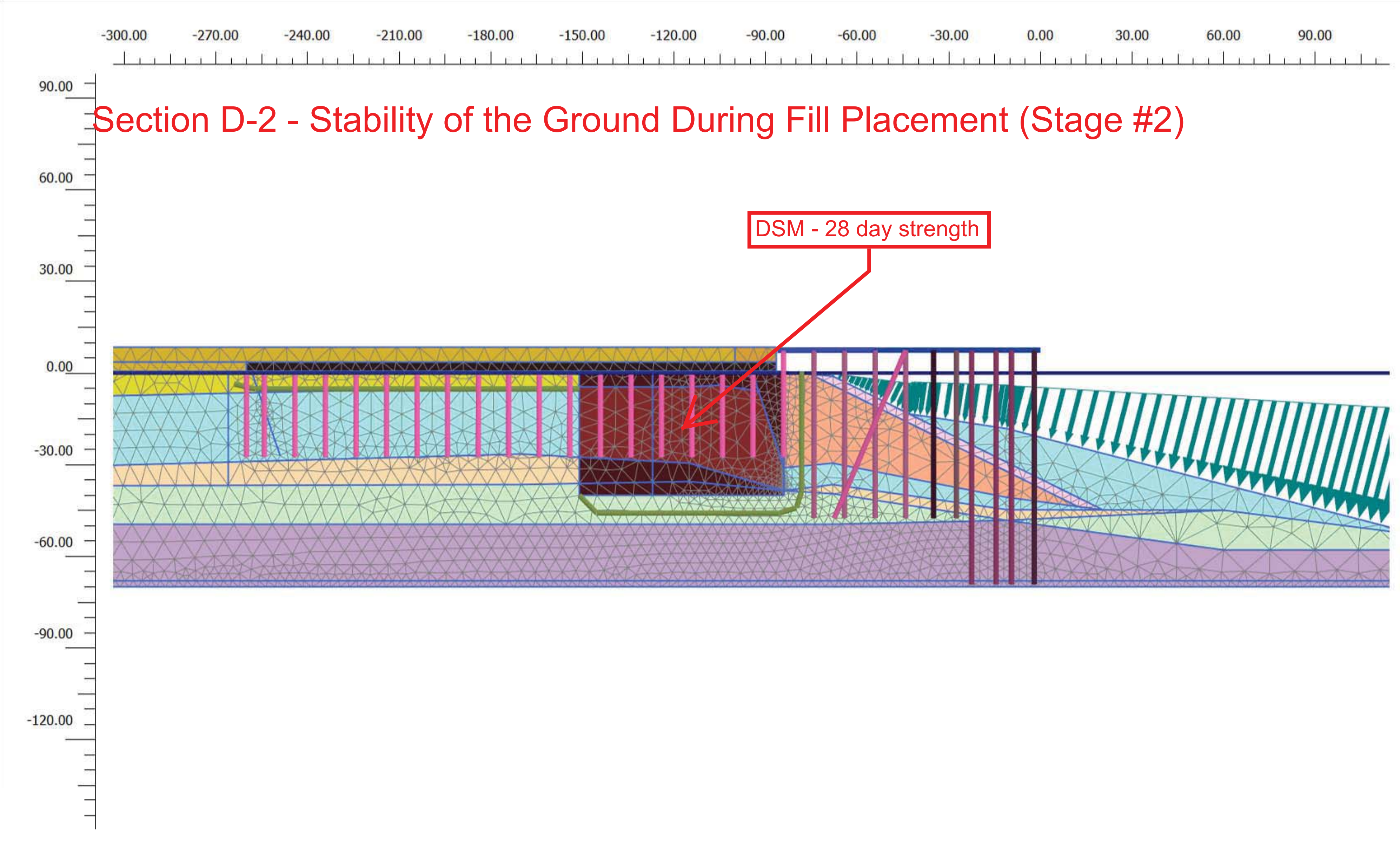
Total displacements u_x

Maximum value = $3.238*10^{-3}$ ft (Element 1414 at Node 4959)

Minimum value = $-0.02501*10^{-3}$ ft (Element 1 at Node 16800)

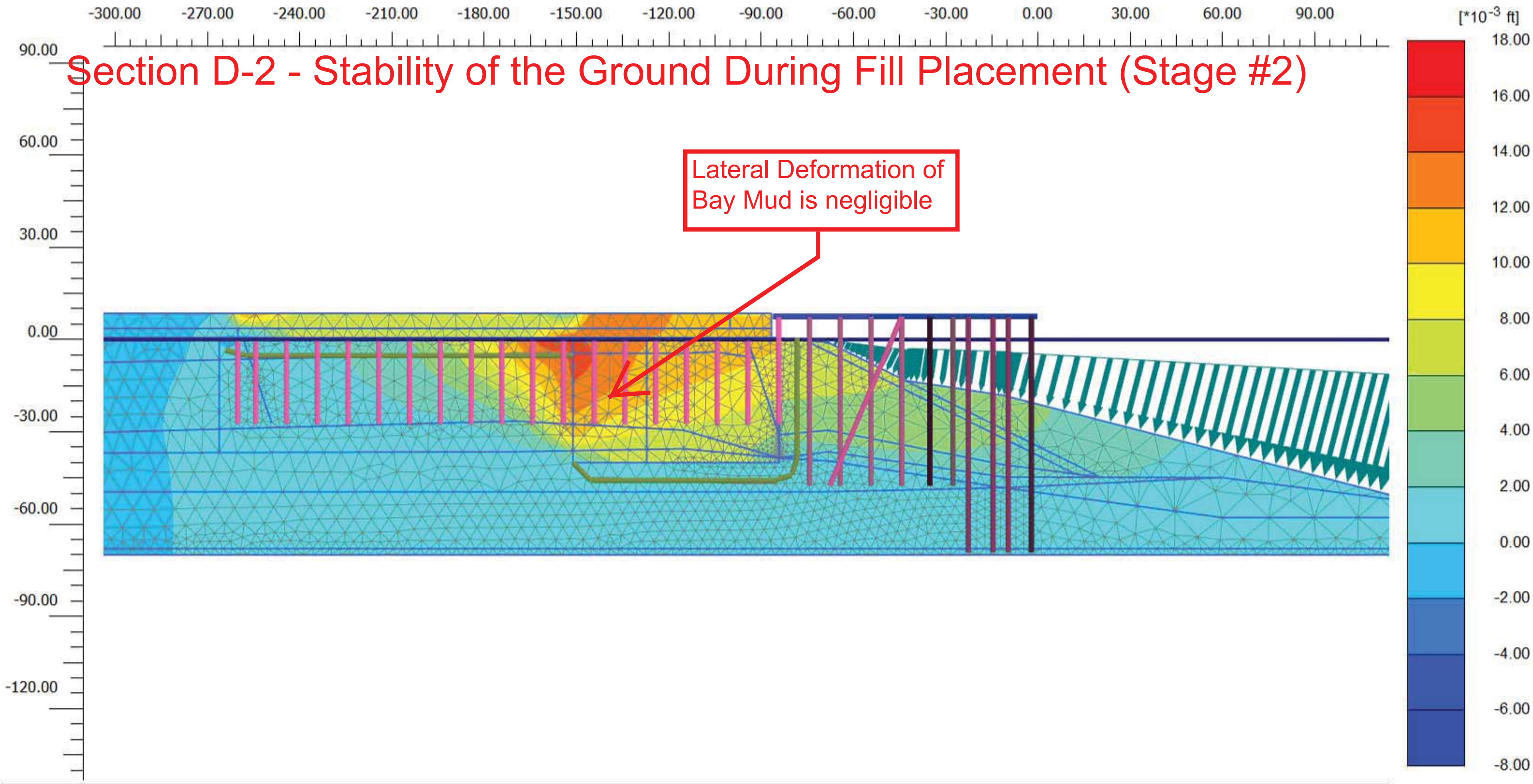
AGI	Project description	Date
		10/21/2018
	Project filename: Section D-2_Stage1	Step: 8	User name: Advanced Geosolutions Inc (AGI)

Output Version 2018.0.0.0



AGI	Project description		Date 10/21/2018
	Project filename Section D-2_Stage2and3	Step 18	User name Advanced Geosolutions Inc (AGI)

Output Version 2018.0.0.0



Total displacements u_x

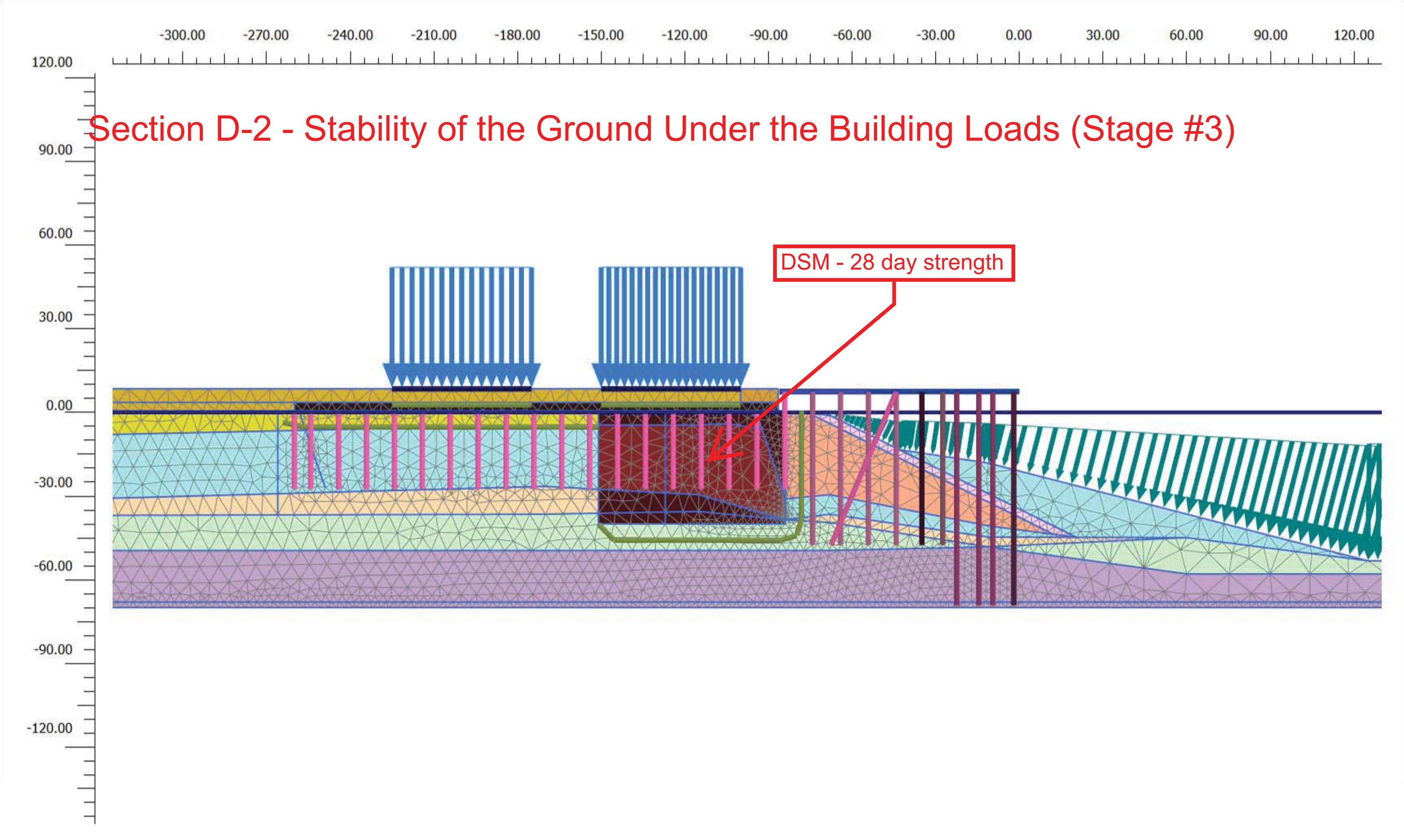
Maximum value = 0.01710 ft (Element 859 at Node 6871)

Minimum value = -6.485×10^{-3} ft (Element 2068 at Node 11214)

Project description	Date
	10/21/2018

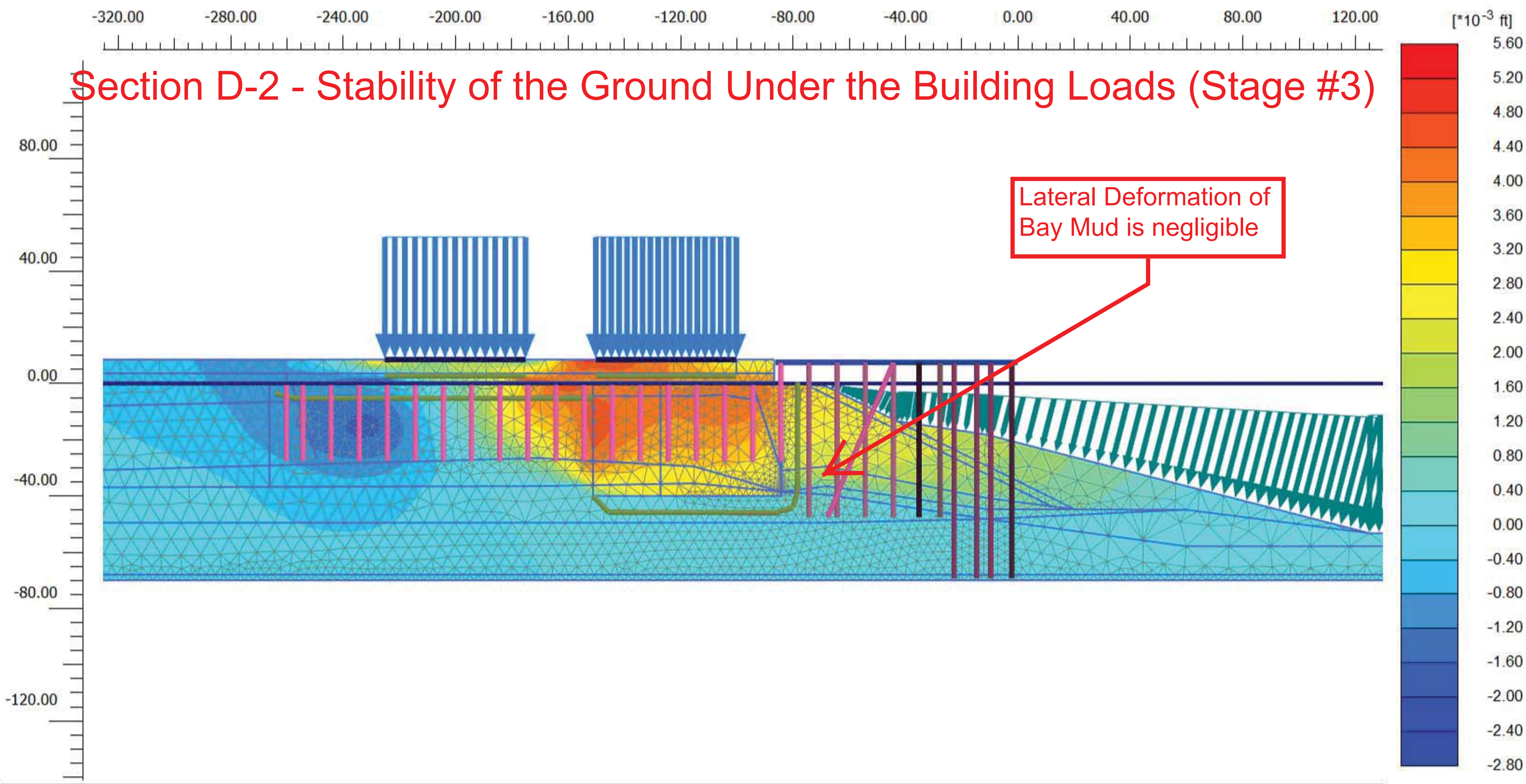
Project filename	Step	User name
Section D-2_Stage2and3	18	Advanced Geosolutions Inc (AGI)

Output Version 2018.0.0.0



Project description		Date
		10/21/2018
Project filename	Step	User name
Section D-2_Stage2and3	7	Advanced Geosolutions Inc (AGI)

Output Version 2018.0.0.0



Total displacements u_x

Maximum value = $5.391*10^{-3}$ ft (Element 859 at Node 6871)

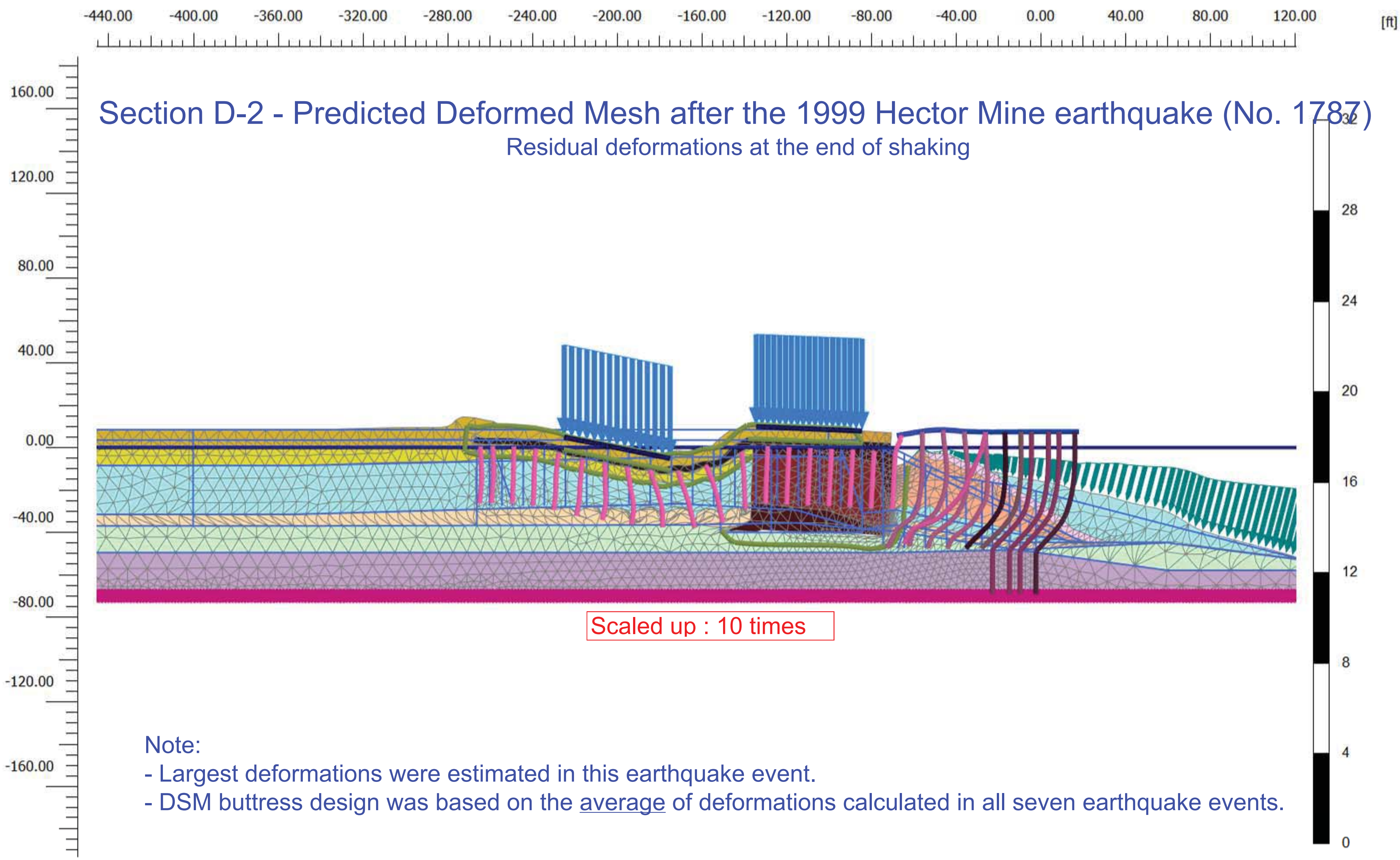
Minimum value = $-2.414*10^{-3}$ ft (Element 355 at Node 8163)

AGI	Project description		Date
			10/21/2018
	Project filename	Step	User name
	Section D-2_Stage2and3	7	Advanced Geosolutions Inc (AGI)

Output Version 2018.0.0.0

Section D-2 - Predicted Deformed Mesh after the 1999 Hector Mine earthquake (No. 1787)

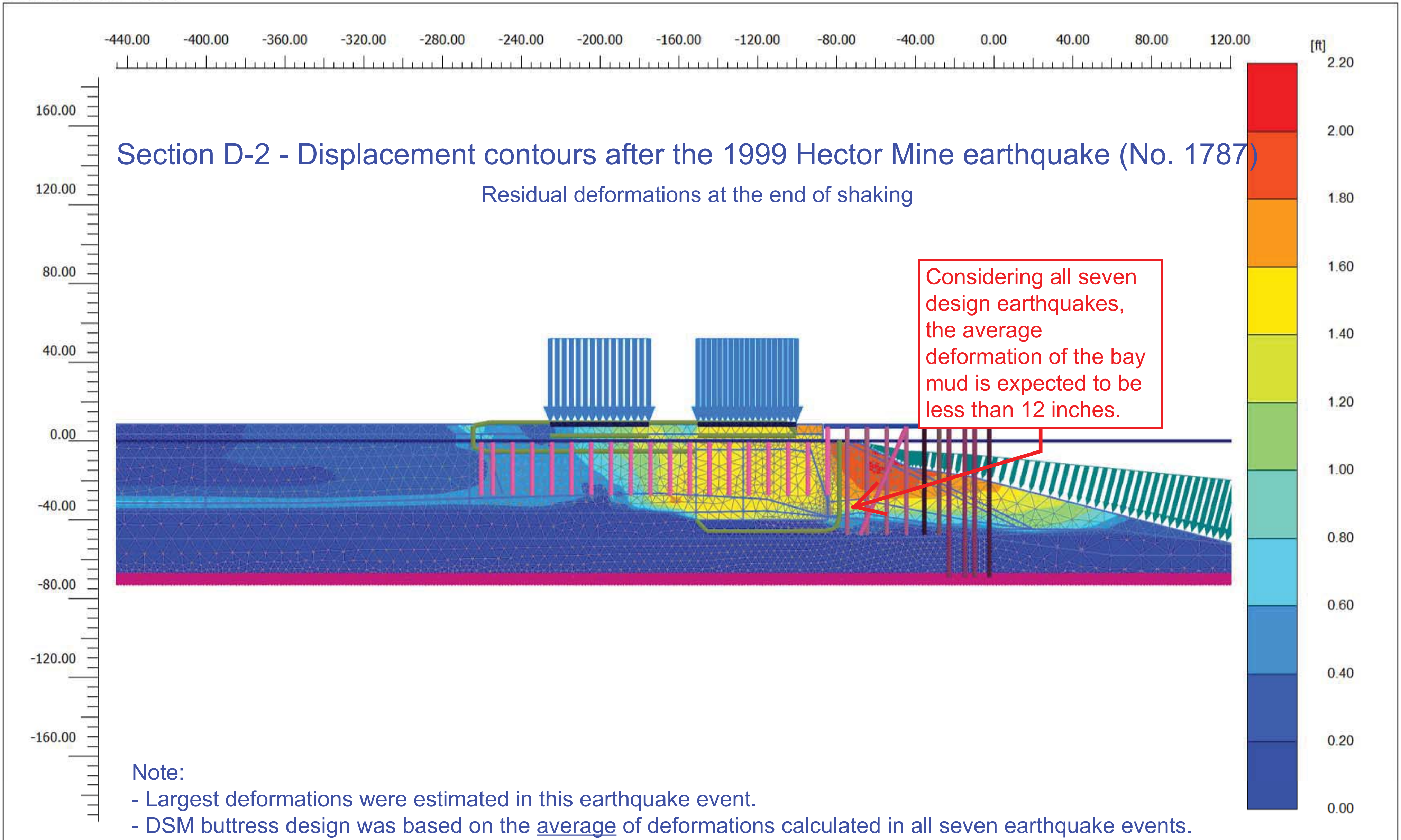
Residual deformations at the end of shaking



Note:
- Largest deformations were estimated in this earthquake event.
- DSM buttress design was based on the average of deformations calculated in all seven earthquake events.

	Project description		Date
AGI			9/5/2018
	Project filename: Section D-2_Dir -X	Step: 5504	User name: Advanced Geosolutions Inc (AGI)

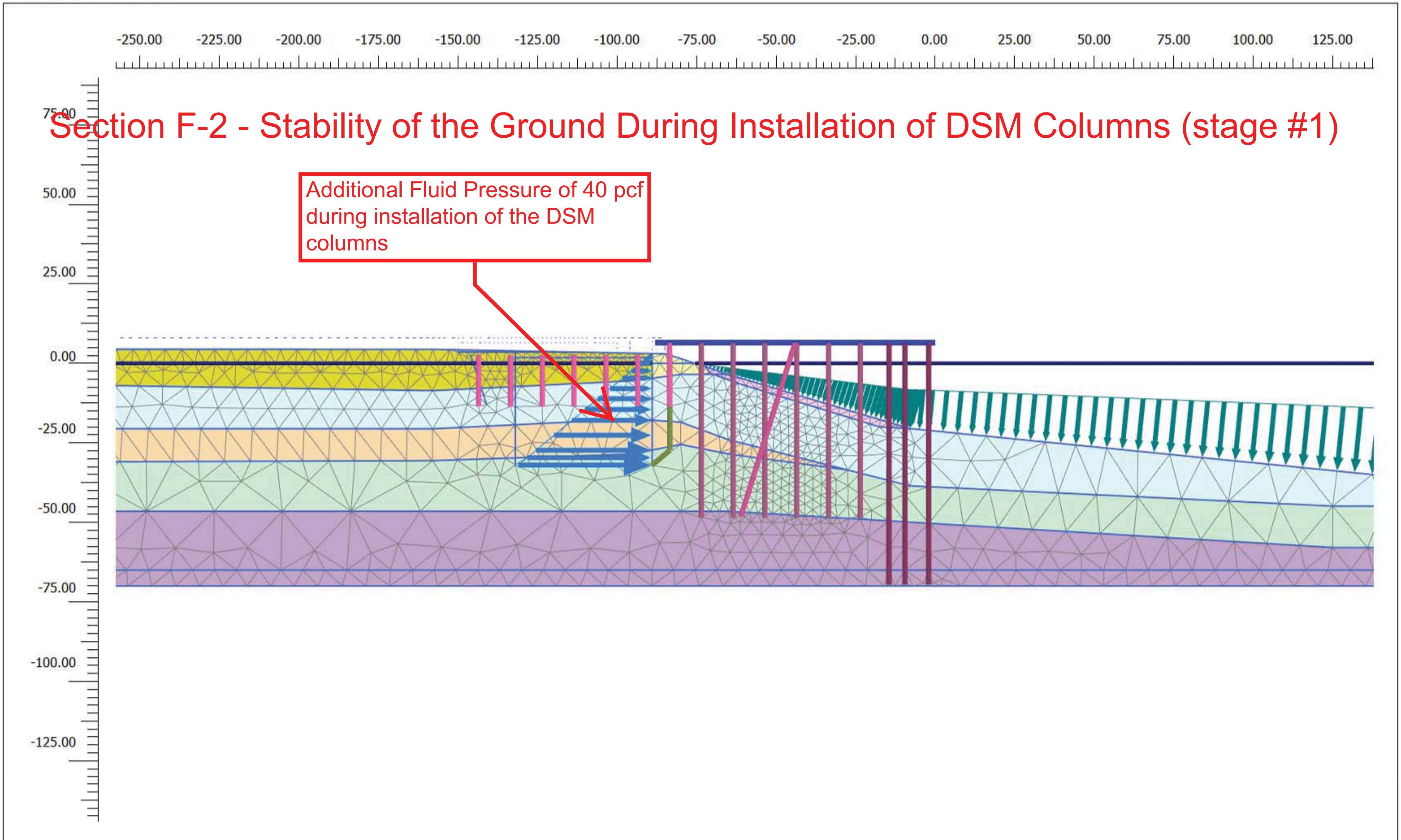
Output Version 2018.0.0.0



Project description			Date
			9/5/2018
Project filename	Step	User name	
Section D-2_Dir -X	5504	Advanced Geosolutions Inc (AGI)	

APPENDIX B

Output Version 2018.0.0.0

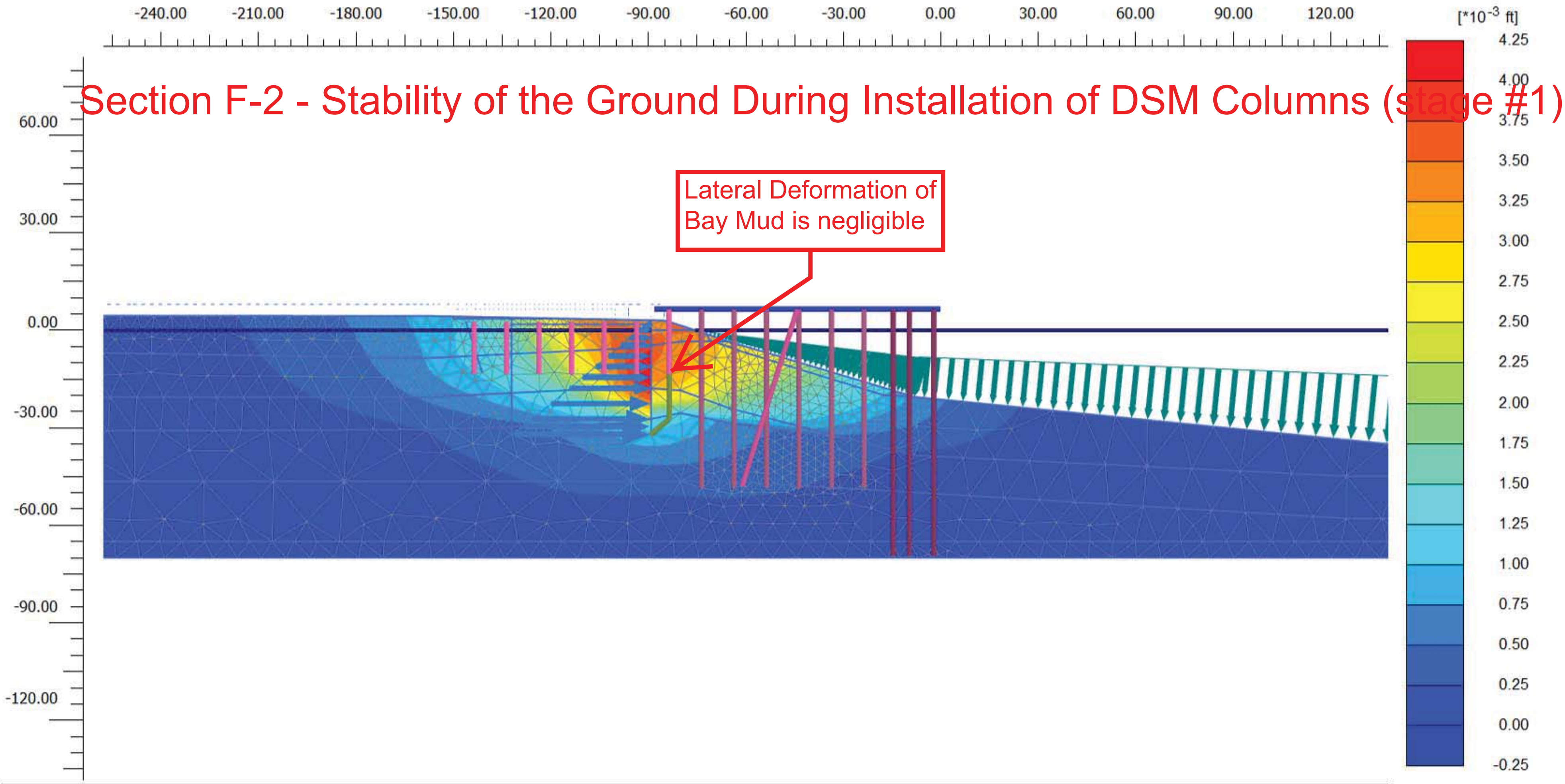


Project description		Date
		10/21/2018

Project filename	Step	User name
Section F-2_Stage1	5	Advanced Geosolutions Inc (AGI)

Output Version 2018.0.0.0

Section F-2 - Stability of the Ground During Installation of DSM Columns (stage #1)



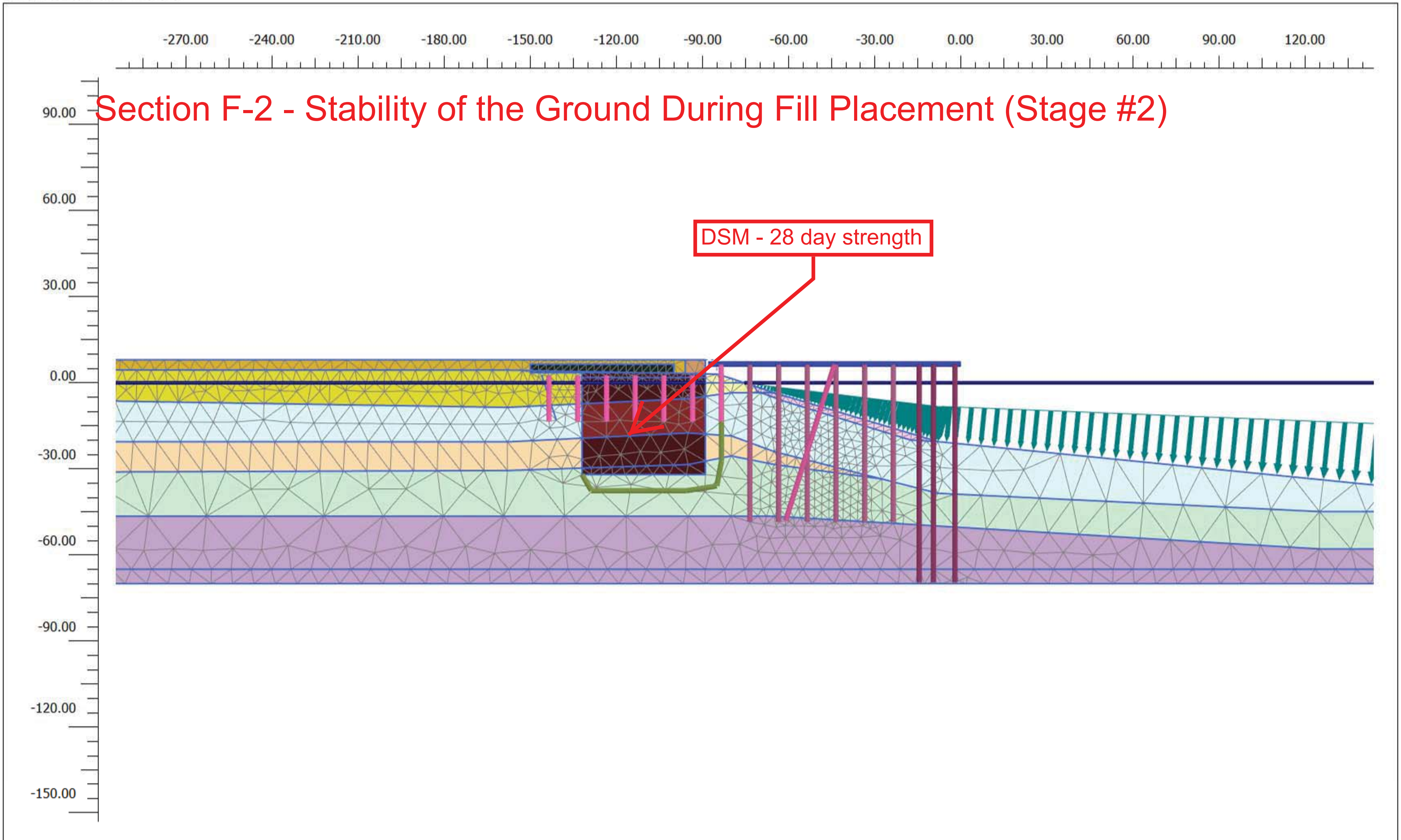
Total displacements u_x

Maximum value = $4.165*10^{-3}$ ft (Element 1937 at Node 6989)

Minimum value = $-0.01104*10^{-3}$ ft (Element 470 at Node 48)

AGI	*Project description*		*Date* 10/21/2018
	Project filename Section F-2_Stage1	*Step* 5	*User name* Advanced Geosolutions Inc (AGI)

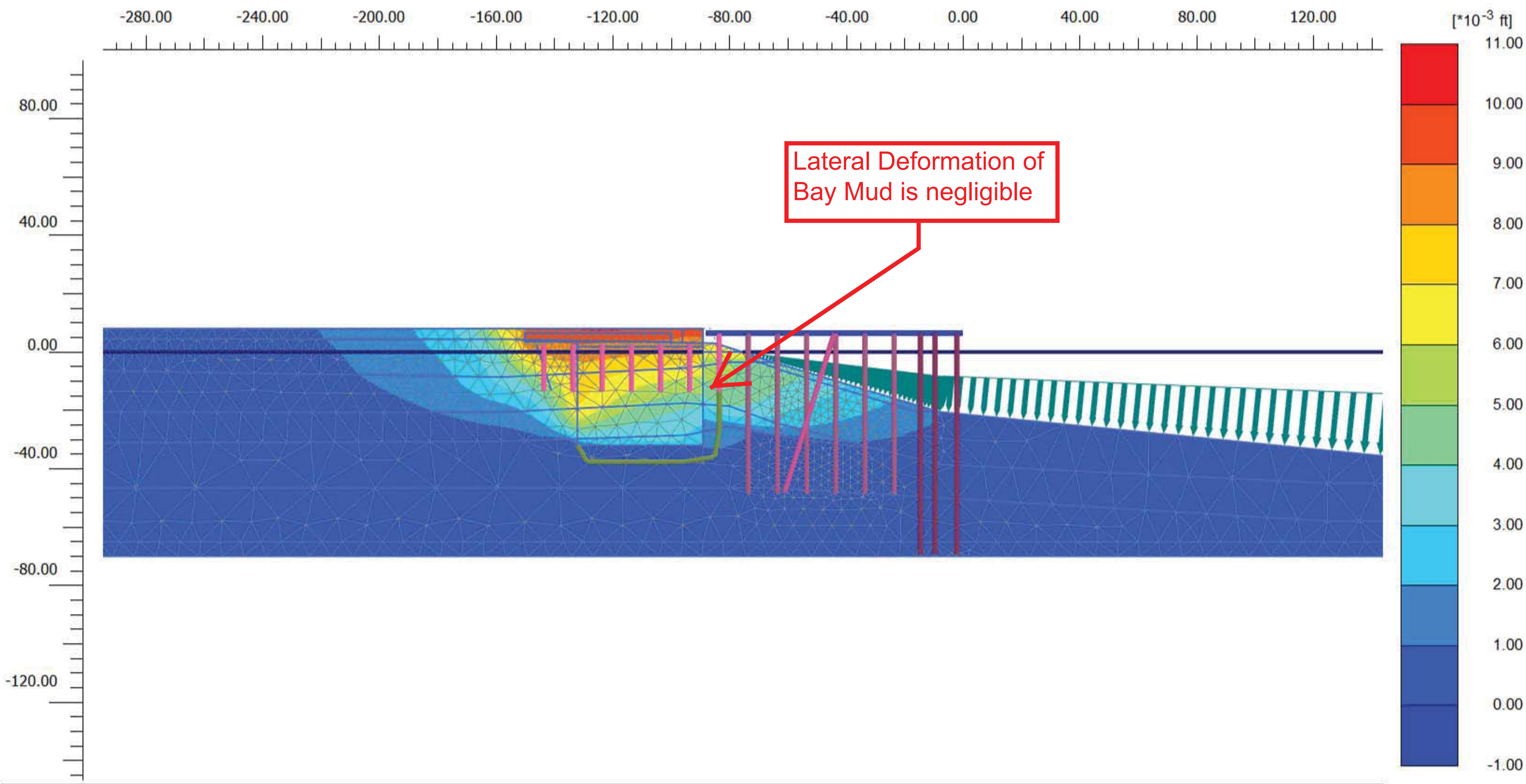
Output Version 2018.0.0.0



AGI	Project description		Date 10/21/2018
	Project filename Section F-2_Stage2and3	Step 13	User name Advanced Geosolutions Inc (AGI)

Section F-2 - Stability of the Ground During Fill Placement (Stage #2)

Output Version 2018.0.0.0



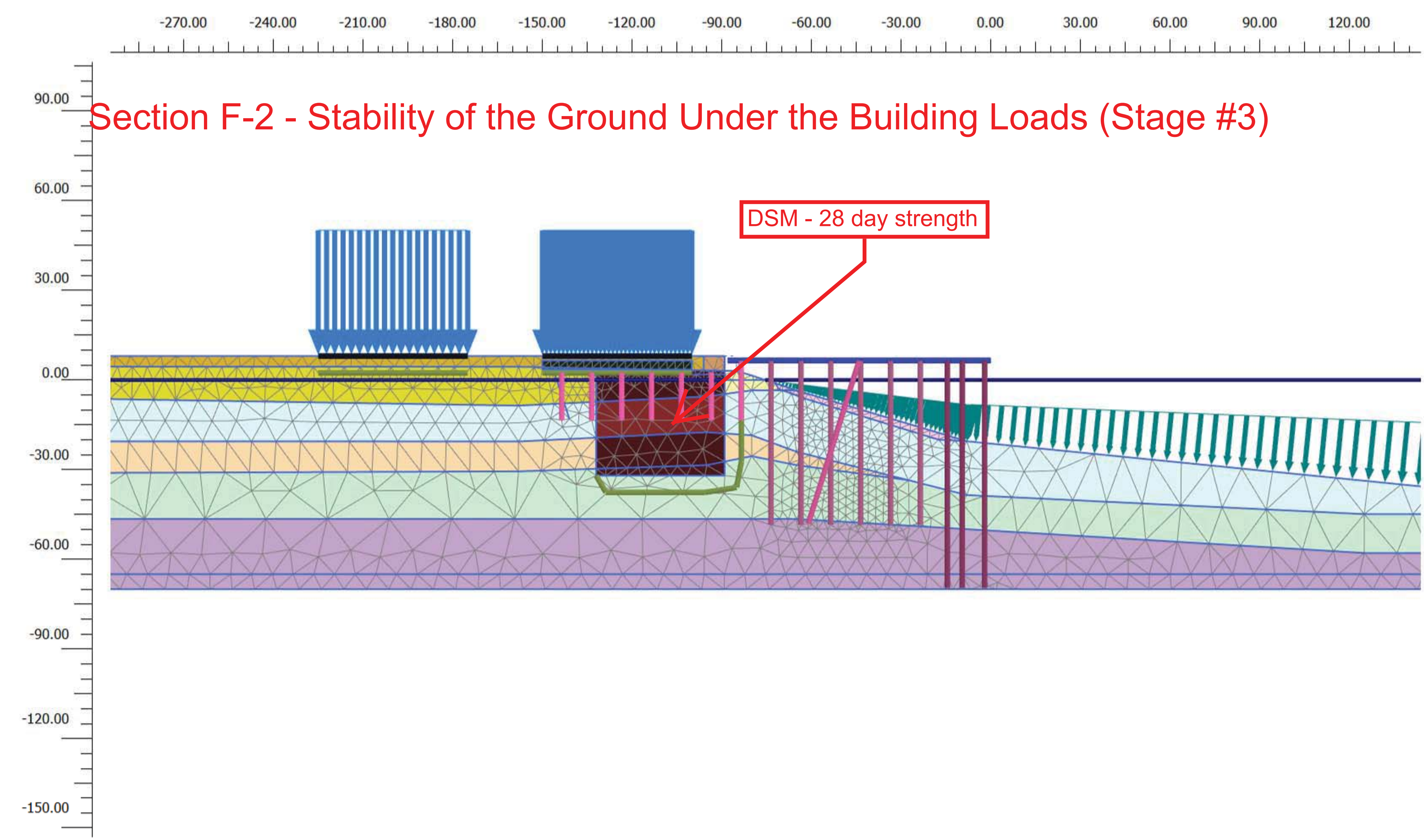
Total displacements u_x

Maximum value = 0.01020 ft (Element 440 at Node 5685)

Minimum value = -0.6329×10^{-3} ft (Element 2043 at Node 371)

AGI	Project description		Date
			10/21/2018
	Project filename	Step	User name
	Section F-2_Stage2and3	13	Advanced Geosolutions Inc (AGI)

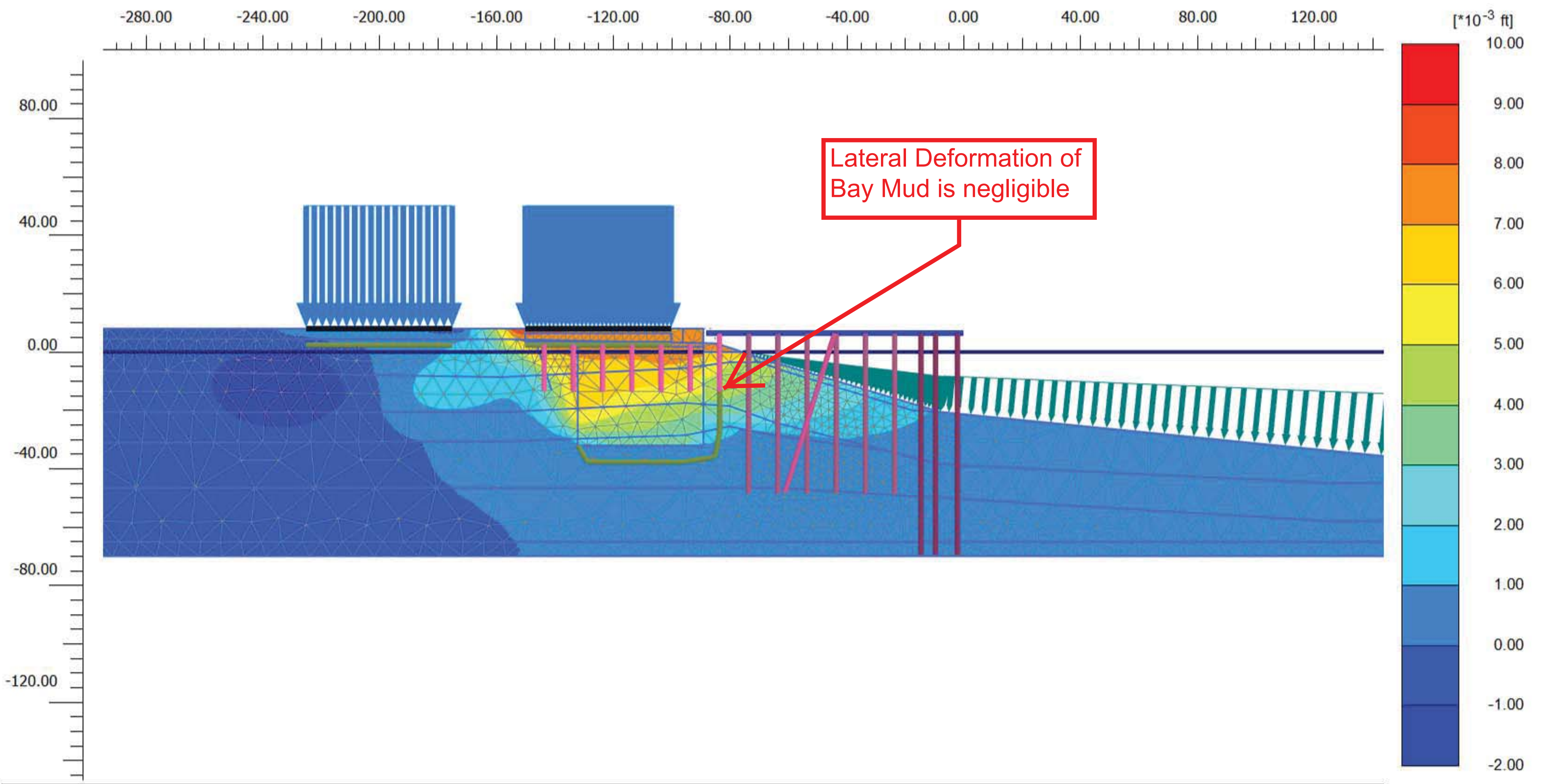
Output Version 2018.0.0.0



Project description			*Date* 10/21/2018
Project filename Section F-2_Stage2and3	*Step* 18	*User name* Advanced Geosolutions Inc (AGI)	

Section F-2 - Stability of the Ground Under the Building Loads (Stage #3)

Output Version 2018.0.0.0



Total displacements u_x

Maximum value = 9.266×10^{-3} ft (Element 168 at Node 5575)

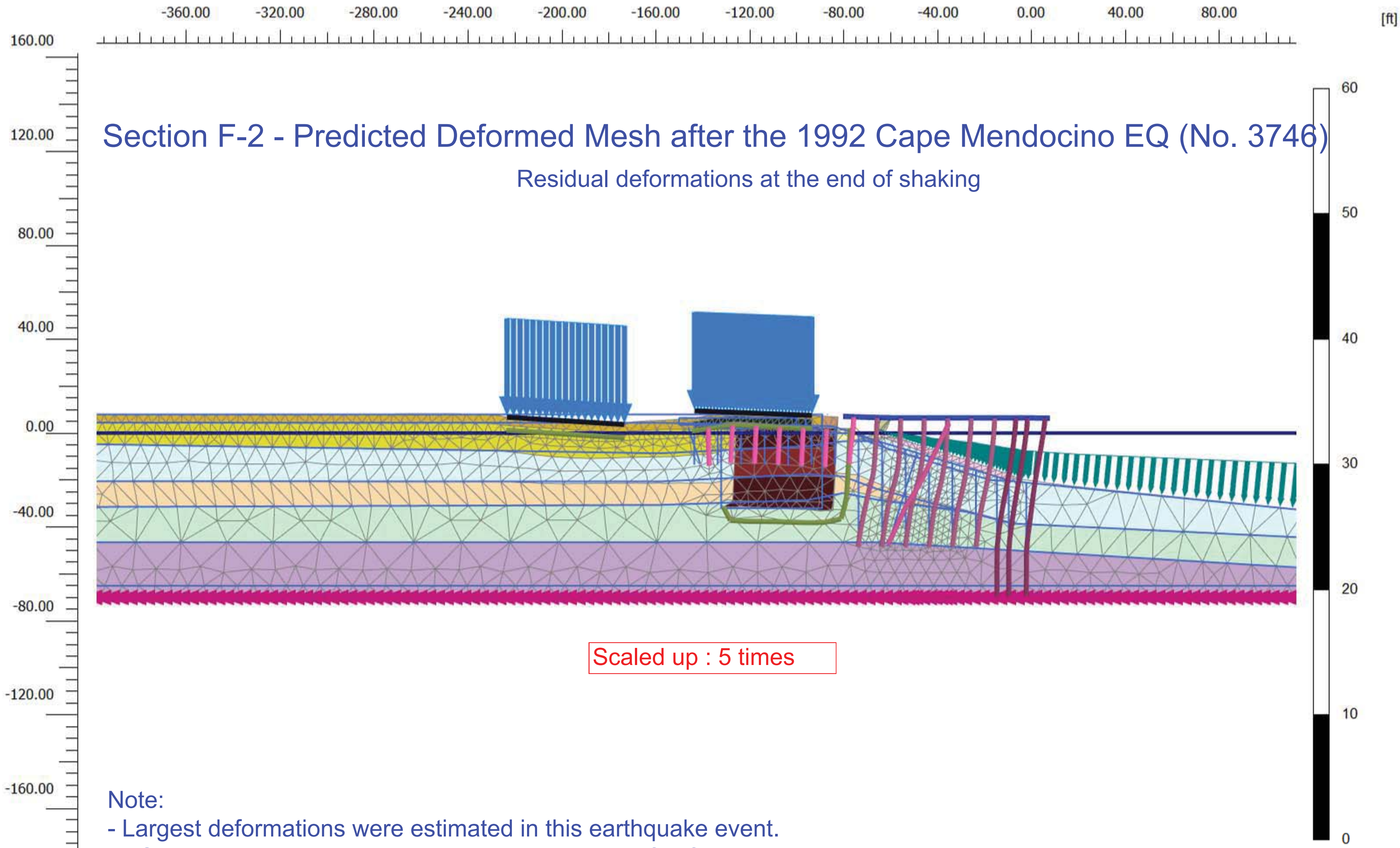
Minimum value = -1.744×10^{-3} ft (Element 1503 at Node 5476)

AGI	*Project description*		*Date* 10/21/2018
	Project filename Section F-2_Stage2and3	*Step* 18	*User name* Advanced Geosolutions Inc (AGI)

Output Version 2018.0.0.0

Section F-2 - Predicted Deformed Mesh after the 1992 Cape Mendocino EQ (No. 3746)

Residual deformations at the end of shaking



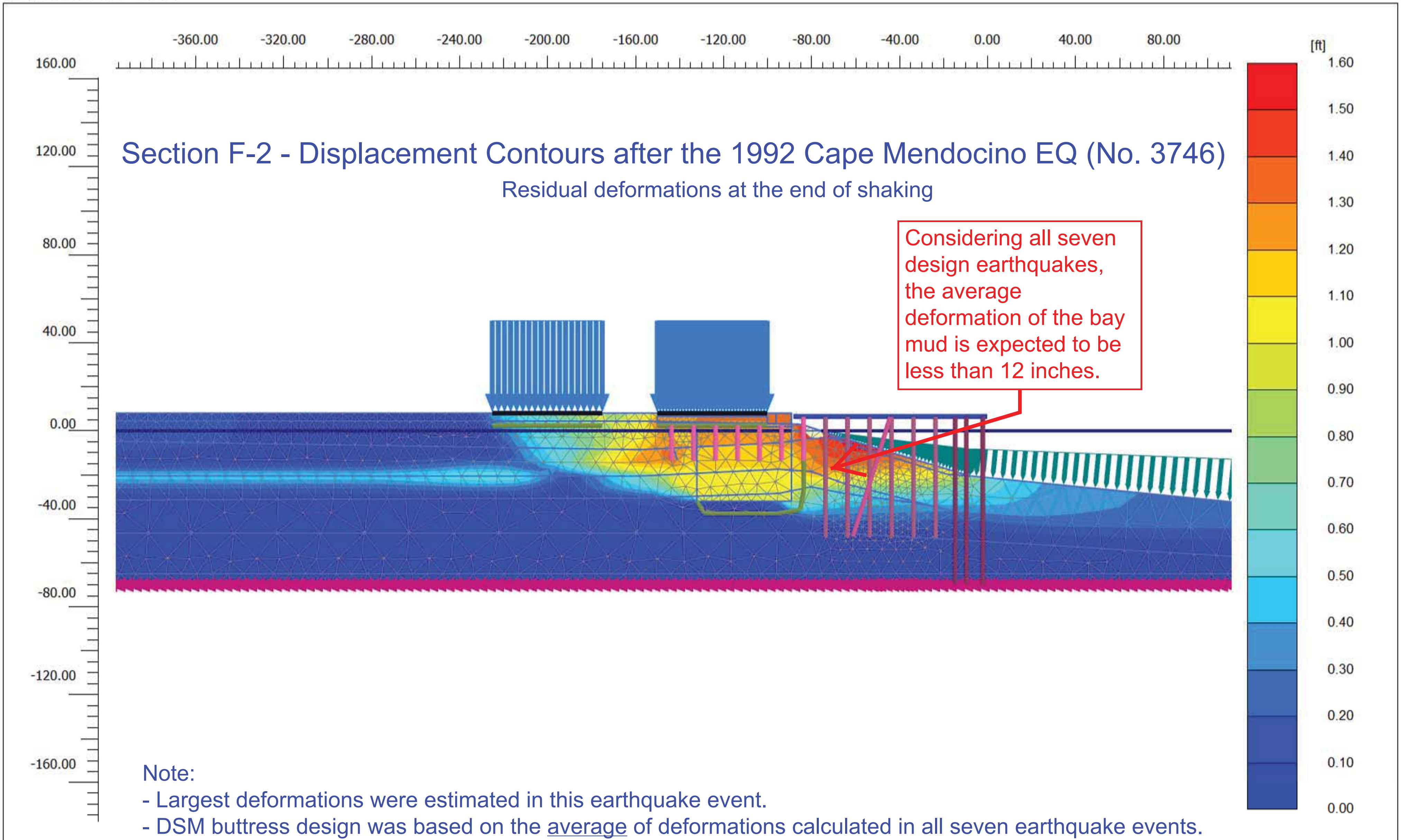
Note:

- Largest deformations were estimated in this earthquake event.
- DSM buttress design was based on the average of deformations calculated in all seven earthquake events.

AGI

Project description			Date
			9/5/2018
Project filename	Step	User name	
Section F-2_Dir -X	28311	Advanced Geosolutions Inc (AGI)	

Output Version 2018.0.0.0



AGI	Project description		Date 9/5/2018
	Project filename Section F-2_Dir -X	Step 28311	User name Advanced Geosolutions Inc (AGI)